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1907 a stock company).

1867. July 16. To the Frenchman Joseph Monier was granted a patent for making reinforced concrete vats.

1868. Additional patent for tanks and pipes.

1869. Additional patent for flat slabs.

1873. Additional patent for bridges.

1875. Additional patent for stairways.

1877. Founding of "Union of German Cement Manufacturers",
(since 1889, Union of German Portland Cement Manufacturers) ²

1878. Introduction of "standards for uniform supply and testing of Portland cement." (Prussian Minister of Public Works).

(1910 Issue of "German standards for uniform supply and testing of Portland cement").

³ 1884. Freytag & Heinschke in Neustadt-a-R (since 1892, Wayss & Freytag), purchased German patent of Monier.

1886. Publication of first theory for reinforced concrete structures in Cent. d. Bauv. (by M. Koenen).

1898. Founding of German Concrete Union.

1904. Publication of "Prussian Regulations for the construction of reinforced concrete buildings."

(1907. Second edition).

1906. Publication of French Regulations for reinforced concrete.

1907. Arrangement of "German Commission for Reinforced Concrete" (the work of the Commission had already commenced in 1904). ¹

1908. Publication of "General Rules for preparing, constructing and testing buildings of rammed concrete."

1909. Publication of Swiss Regulations for reinforced concrete.

1910. Publication of "German Standards for uniform supply and testing of steel Portland cement."

1910. Introduction of "General supply conditions of the members of the German Portland Cement Manufacturers belonging in Germany."

1911. Publication of the Austrian Regulations for reinforced concrete.

Note 1. p. 2. The yearly product of this manufactory with 14,000 H. P. machinery amounts to 2,500,000 barrels. This is the greatest quantity of cement produced in Europe by a single company.

Note 2. p. 2. To this Union belong by far the most Portland cement manufacturers in Germany. The members have bound themselves under high penalties to supply only Portland cement to the trade, that is free from all adulteration. The products of the members of the Union^{and} tested annually in the laboratory of the Union at Karlsborst near Berlin in every way to maintain the penalties; the results of the tests are made known in the general assembly. -- The total production of the Union in the year 1913 amounted to about 40,400,000 barrels of Portland cement. (Each 170 kil net weight).

Note 1. p. 3. The existing means provided for the scientific experiments of the Commission amounted to 326,000 marks. Of this was used 320,000 marks up to the end of 1913.

1. Safety against fire.

This is caused by the bad conducting power of the concrete. The use of steel by itself affords no sufficient protection from fire, and even in most cases fires cause entire destruction in consequence of ^{the} becoming red hot by the heat. The heavily loaded steel supports, whose bearing capacity generally disappears at a heat of 600° to 800° C, producing by its fall not only a general destruction of the parts beneath it, but ^{also} in most cases the overthrow of the adjacent masonry. All salvage work and putting out fire are naturally made much more difficult; all close fighting of the fire is made impossible on account of the danger from falling. The present rules of the building officials therefore require a covering of the steel parts for the purpose of obstructing the direct effect of the heat on the steel member. Coverings of concrete, as such are frequently in use, increase the cost of building in a considerable measure, since in most cases it merely represents dead weight in statical respects. ¹ An exception is formed by columns of enclosed cast iron. (See Section V. c).

Note 2. p. 3. Mild steel loses at about 500° C more than half its strength, already at 600° C three-fourths, and it melts at about 1400° C. But at 1000° C it is already entirely soft.

Note 1. p. 4. Even covering with hollow tiles and terra cotta afford no fire protection free from objections, since if once heated and then subjected to a stream of cold water, these mostly break, and then can no longer protect the steel from the flames. Likewise floors with I-beams having concrete panels

tamped between them, but with unprotected flanges at bottom, ~~cannot~~ cannot be termed fireproof. Such constructions cannot pretend to the name of reinforced concrete floors.

On the contrary in reinforced concrete structures the enclosing concrete is the proper material, that requires no further protection from fire. It is in a high degree capable of offering a successful resistance to all injuries by fire. Danger of falling is absolutely impossible, which results in a substantial reduction of all salvage work and extinguishing of fires. The floors can also support loads in fires greater than the ~~live~~ live loads. Even extreme injuries by fire can be limited to the story, where the fire broke out, by properly constructed floors of reinforced concrete. Other rooms remain without danger; ~~the~~ the combustible objects in them are not attacked by fire. The penetration of the water into the story beneath is effectively prevented by ~~the~~ linoleum, covering a sufficiently thick basis of asphalt. ² The restoration of the work may proceed rapidly, after all parts affected by fire and water have been tested for strength by loading tests, and have been sufficiently tried by tapping with a hammer. ¹ Even with the unprotected cast iron supports the extremely dangerous sprinkling with cold water ~~will~~ will not affect the reinforced concrete injuriously. Since the formation of a crack in the concrete does not usually occur, neither a direct effect of the red heat nor sprinkling the water on the steel parts does ~~take~~ take place. ² Many experiments have proved, that the usual layer covering the steel with a ~~thickness~~ thickness of 1 to 2 cm already suffices to secure the permanence of a reinforced concrete floor in an average fire. ² The ~~heat~~ heat penetrates but little, and in any case produces superficial injuries. But a destruction of the whole is scarcely to be feared, even by a continuing action of the flames; this will only cause the plastering to flake off. Also facing concrete is in general not advisable against the effect of flames and quenching water (sandstone is destroyed thereby). -- The measure of safety of reinforced concrete walls can be placed 2 to 4 times as great as that of equally thick brick walls; for the same resistance a brick wall 38 cm thick may be replaced by a reinforced concrete wall of about 12 to 15 cm. in thickness. ⁴

Note 2. p. 4. See Burning of Kaders' Warehouse, Dresden. B & E. 1911. p. 253. -- Arm. B. 1911. p. 369.

Note 1. p. 5. In calculations of rentability the question of security against fire -- one considers the insurance premium -- is of an importance not to be underrated. While in one case perhaps 4 or 5 p. c. is to be paid in premiums, it is sufficient here with the use of reinforced concrete throughout to pay only about 1/4 to 1/2 p. c.; for example, that makes for the larger warehouses a saving of many thousand marks annually.

Note 2. p. 5. If cracks are formed, these are mostly unimportant, since they generally close again after cooling. In unfavorable conditions results an occasional falling of the plastering and concrete to leave the steel exposed, caused by the expansion of the water in the concrete changed into steam.

Note 3. p. 5. If in the plastering it is intended to be entirely fireproof (for example in vaults), then naturally are employed floors thicker than those mentioned above. (3 to 5 cm).

Note 4. p. 5. See "Fireproof Walls of Reinforced Concrete". E & S. 1910. p. 346.

The fireproof quality of the structure naturally assumes a thoroughly appropriate architectural construction. In regard to the slowness of transmission of heat and of permanence of the covering of the reinforcement in the fire and in resistance to the quenching water, preference is to be given to a concrete of limestone spalls to one of gravel. There is also advisable the use of cinder and pozzolane concrete. In any case the concrete must be sufficiently old; if too fresh, concrete crumbles. Further valuable is a good end connection of the reinforcement.

By laboratory experiments and fire tests (for example the Vienna model theatre) men have obtained the knowledge, that in reinforced concrete has been found a building material fireproof in every respect, as well as first proved by many practical experiences in great conflagrations. Recall particularly the fire in the city of Baltimore, where about 2500 houses were destroyed, but all reinforced concrete structures were uninjured. The same appeared in the earthquakes and fires in San Francisco in 1906, and in Messina in 1908. For improvements may further be employed a "mortar covering". (Clay. 1913. No. 32). Finally it may also be mentioned, that at the instance of the Central Union of German Cement Products and Artificial Stone Manufacturers, granite and concrete steps were tested for the capacity of resistance to fire. While granite steps -- in per-

serve the following brief description of a fire test; two lattice steel girders, Vissintini system, 8 days old, calculated for a live load of 250 kil/m^2 , span 5.0 m, height 24 cm, with a load of 800 kil/m^2 , were heated by a strong wood fire to 1000°C , during the fire being played on with cold water, then cooled and finally loaded with 1281 kil/m^2 until they broke. With a load of 1194 kil/m^2 appeared a deflection of 18 mm. After the fire the lattice girder (thus before the loading test) showed neither any cracks nor a measurable deflection. ²

Note 1. p. 7. Such experiments chiefly extend to:--

- a. Different methods of reinforcement.
- b. Most suitable aggregate and mixing proportions.
- c. Reduction of strength during and after heating, (with rapid and slow cooling).

Also see German Commission for Reinforced Concrete, Rept. 11, "Fire Tests of Reinforced Concrete Structures." 1911. Wm. Ernst & Son. Berlin. (Arm. B. 1911. p. 445).

Note 2. p. 7. Experiments in the Laboratory for Clay Industries indeed showed at 1000°C a reduction of resistance to compression 1:3 of 220 to 75 kil/cm^2 and a loss of tensile resistance of 22.4 to 1.4 kil/cm^2 . T. Brut even -- for the same temperature -- proved a loss of average compressible resistance from 374 to 47 kil/cm^2 . Yet if one takes into consideration, that an injurious temperature of very high temperature in an experimental laboratory lasts but a short time in most cases, and because the bad conductivity causes the heat to pass very slowly to the steel. According to later experiments the compressible resistance of concrete exposed to fire is reduced to about 0.4 of that at the beginning.

even for vaults is now preferably employed concrete, strengthened in a proper manner by inserted steel bars (grillies). In all cases it resists melting temperatures better than a covering entirely of steel, is proof against thermite and any cutting & curdler of the burglar. (See Contrb. 1912. p. 139).

See further the Essay, "Fire and Load Tests of a Reinforced Concrete Floor", B & E. 1909, p. 326 Also see the remarkable statements concerning fires. B & E. 1911. p. 306, 322. (Monastery at Hamburg), as well as the publication of German Concrete Union; "Fireproof qualities of Concrete, Reinforced Concrete, Steel and Wood." 1911. Berlin. Also "Fireproof Quality of Re-

Reinforced Concrete in the greater fires in the year 1911," and Series II, 26 of the German Committee for Reinforced Concrete.

2. Security against Rust for the Reinforcement.

Likewise the fear, that the steel would rust by the wet concrete applied to it, is now entirely removed. On the one hand the bond resistance proves the mechanical connection of steel and concrete, so on the other the lack of formation of rust a sufficient proof of a chemical influence of the cement on the steel. It appears that after the close covering prevents contact of air, a double silicate is formed, which then represents the protecting covering of the steel.¹ Experiments continuing for years and observations of reinforced concrete structures of the most different kinds in the fullest measure have proved, that the water contained in the plastic cement cannot rust the reinforcement. There even prevails no further doubt, that the steel is protected from the formation of rust by plastic concrete (mixture with sufficient water) better than by any other material.² The bluish rolled surface is still to be distinguished after years. The mixture of the concrete must be thorough, the addition of water not too great, since otherwise the resistance of the concrete would be reduced. There is taken for strongly stressed structural parts (floors, beams, supports, etc.) at least a mixture of 1:5 or 1:6 and not less. With leaner mixtures (for example, for foundation footings and slabs) there is recommended a preliminary coating of the steel bars.¹ In no case should the steel rods be set too close together, since then they would not be sufficiently covered by the concrete. Lime and gypsum mortar will attack the steel and cause the formation of rust.² Even hair cracks in the tension zone cannot affect the safety against rust, so far as the influence of the atmosphere does not come in question.

Note 1. p. 8. According to Dr. Schönlank the actual cause of the protection from rust is to be sought in the fact, that on the one hand the steel, and indeed all except noble metals, are not attacked by strong alkalies, on the other the cement in being mixed with water, exhibits a strong alkaline reaction, in consequence of the hydrolytical splitting of the lime. (Contrib. 1911. p. 148). He opposes the opinion, that the nonoxidation of the steel on concrete only results from the fact, that the air, and thus the access of oxygen is entirely excluded.

Note 2. p. 8. In America in the laying of very large steel water pipes, an external and an internal coating of these with concrete has been applied with good results. E & E. 1913. p. 91). Likewise for chimney cooling structures, a concrete covering of the steel framework in the interior as a protection from rust has been employed with advantage.

Note 1. p. 9. See Section II, C.

Note 2, p. 9. It has been established by experiments, that the addition of slag (containing iron oxide) and of cinders (containing sulphur) no satisfactory protection from rust can be obtained.

On the part of the Royal Upper Building Officials in Munich, from a ~~Monier~~ ^{Monier} Bridge, 17 years old, and exposed to heavy traffic, (city streets Reichenhall-Jellendorf), were taken samples from the soffits of the arches (3 arches of 20 m span and 25 cm thick at crown). Where fine grained concrete existed, the steel was entirely free from rust; only in the coarse concrete appeared small spots of rust, that could easily be rubbed off with the finger.

To similarly favorable results came Wayss & Freytag Co. They had small pieces cut out and replaced anew in a sewer for use for 11 years. The reinforcement was entirely free from rust.

Furthermore at the Provincial Exhibition in Nuremberg in 1906, a reinforced concrete arch was constructed, for which rusty steel was employed. After the close of the exhibition, about one year after its construction, the arch was loaded until it broke. On breaking up and cleaning, it was then found, that the previously very rusty steel had become entirely bright. Dr. Rohland (Stuttgart) has proved this by experiments on a small scale. Steel bars coated with iron oxide, after being long in contact with setting and hardened cement, were broken out of this free from rust. Dr. Rohland conjectured, that this condition was based on the effect of the hydraulic lime hydrate separated during the setting, on the iron oxide. ¹ The process of removal of rust was aided by the gypsum contained in the cement. The removal of rust occurred on all parts of the steel, that were in the ~~closest~~ ^{closest} contact with the cement.

Note 1. p. 10. See Rohland, "Removal of Rust from Steel in Reinforced Concrete." Stahl & Eisen. 1908. No. 11. Further contributions. 1911. p. 148. Mittheil. of Society of German Engineers.

no. 28. Concerning protection from rust, also see 2 & 3. 1911. p. 406, 504. Ann. B. 1912. p. 200.

3. Capacity for Resistance to Smoke Gases.

It is known, that many building materials, like steel, stones containing lime and other materials, are strongly attacked by smoke gases on account of the sulphuric and carbonic acids contained therein, particularly when the gases have a high temperature. Most inconvenient is a frequent interruption of the * works, a great percentage of water content, and great variations of temperature of the gases of combustion. The fear that also the concrete under the effect of such gases might deteriorate, is therefore not necessarily to be rejected, especially since carbonic acid and sulphuric acid cannot be counted with the best friends of concrete. But still many experiments and manifold practical experiences have produced the contrary evidence with objections. Very notable in this respect are the experiments of the Austrian K. K. Southern Railway; samples were taken from a Monier bridge 13 years old, indeed from places daily exposed to smoke gases from locomotives passing beneath. The samples were then tested chemically, and it was shown, that the concrete always still possessed a great strength, and there appeared not the least trace of permeability by water. Steel reinforcement and concrete firmly adhered to each other. The bearing members, like the small wire bands, were free from rust and appeared faultless, although the covering coat of concrete was only 2 to 3 cm thick.

For the Saxon Railway Administration, from 1892 to 1901, some 14 great heating buildings were equipped with monier smoke flues, which while the engines were heated had to convey the smoke from the locomotives to high chimneys. Even for carrying off the fire gases from the smiths' fires in the locomotive smith shop, where a temperature up to 400° C occurred, the reinforced concrete came into extensive use. All pipes were in excellent preservation after the lapse of many years; it was even determined that for a higher temperature existed a greater resistance of the concrete; the covering of the steel amounted to only about 2 cm.

The entirely rebuilt Portland Cement Factory of the Bonn Mining & Foundry Co was entirely built of reinforced concrete, and among other things also exhibits a reinforced concrete ch-

chimney 60 m high with a dust chamber also built of reinforced concrete, in which generally prevailed a temperature of 500° to 600° C. In spite of the strongest vibrations by stone crushers, no leakage has yet been found from this chamber. The protecting covering of the reinforcement here is indeed 5 cm. For similar structures the smoke ducts and chambers, there is advisable also a coating of fireproof clay or of petroleum asphalt; the covering of the steel may then be thinner.

4. Rapidity of Construction.

The raw materials are delivered in a simple way, and prepared by workmen in the shortest time. Gravel, sand and pebbles are found everywhere; cement is also rapidly produced, and round rods are usual in commerce, always kept ready in great abundance by dealers. Reinforced concrete construction is then employed in all cases with great advantage, where extensive structures are concerned, that must be constructed in a relatively brief time. For the steel construction-- especially in very busy times, long delays in delivery from the rolling mill are to be considered.

5. Adaptability, economical use of space, great Bearing Strength.

Reinforced concrete is suited not only to regular, but also to all irregular forms. There may be recalled here on the one hand theatre buildings, on the other restorations of ruins,¹ fallen art monuments, etc. All shapes are indeed limited by the relatively high cost of the forms. Art and cut shapes are readily produced, and also complex vaults and stairways are constructed. Then also the economically correct estimation of the peculiar resistance of the combined materials permits an important reduction of the structural height, as well as a wider spacing of supports, for even with the existence of heavy loads, the floors may be of very wide span as a result of their great strength. The number of columns required will thereby be reduced to a minimum; so that the interior gains substantially in visibility, the ventilation and the light are also better. Just for the rooms, which are exposed to certain limitations in plan and height, have the advantages mentioned a peculiar importance. By the monolithic character of the whole, especially by the arched floors is produced great security against sidewise forces.

Note 1. p. 12. For example, certain parts of the ruins of A Heidelberg Castle have been supported by reinforced concrete.

Reinforced concrete also permits the undertaking of extended footings for defective foundations of structures. See Part II, 7th edition, p. 151.

Likewise too little account is taken, that the resistance of the concrete may quite considerably increase with age in some conditions. Men depend entirely too much on the known resistance of a cube after 28 days, and do not consider that this resistance is often twice as great after the lapse of some years. Even after 15 years an increased resistance may be proved. (Also see Section III. F). According to the statements of Dyckerhoff a concrete of 1:6:10 after 10 years attained a resistance to compression of 233 kil/cm², and a concrete of 1:8:13 in the same time a resistance of 217 kil/cm². At the Munderking bridge over the Danube, breaking experiments showed a resistance of concrete after 28 days of 254 kil/cm², and after 2 2/3 years, 520 kil/cm². It also appears authorized to consider a smaller factor of safety in reinforced concrete, than is mostly prescribed by officials, -- as Professor Engesser rightly says -- so that annually a great amount of the cost is applied uselessly, and the means of the people are diminished. On the contrary in steel structures the safety can neither increase nor diminish. ¹

Note 1. p. 13. The factor of safety of reinforced concrete structures as a rule is taken higher than in pure steel structures, as particularly shown by Professor Schüle. (B & Z. 1910. p. 332).

It is also to be considered, that by neglecting the tensile resistance of the concrete, as well as by the rigid connection of the whole, the actual factor of safety is generally higher than is obtained by calculation.

Also the resistance of concrete to wear, which plays a part in the production of cement slabs, steps, floors, etc., is entirely satisfactory, even frequently better than granite. It is notable, that cements with high tensile strength are not well suited. Experiments at the Materials Testing Laboratory at Gross-Lichterfelde with the grinding machine (440 revols.) have shown, that the use of crushed hard stone for the surface of the concrete is advantageous (granitoid slabs). Frequently

an addition of iron fillings to concrete has been made with advantage. (See Part II. 7 in edition. p. 95.).

6. Resistance to Vibrations and Shocks.

The vibrations of rapidly moving machines exert no injurious influence, as fully proved by many experiments and experience for years. The fear that the reinforcement would be gradually loosened by such vibrations, and the entire structure thus be gradually destroyed, is unfounded. In consequence of the extraordinary elasticity of the structural materials also the shocks occurring in railway bridges are better received by concrete arches strengthened by steel, than by arches of any other structural material, in spite of the great mass of concrete ~~is~~ used to a considerable extent. The effect of shocks at the ~~the~~ places concerned is not increased in any particular way, but is transferred to all parts of the structure, so that fracture or crumbling of the building is excluded at that place. There occurs only a trembling of the whole; the living force of the shock is exhausted in this manner, and no injury to the structure occurs. Just as this power of resistance to shocks of ~~a~~ every kind again offers a substantial advantage in a fire; for the fear that a reinforced concrete floor would be destroyed by falling articles, parts of machines, goods, etc., mostly lacks all foundation. For foundations of machines is to be ~~be~~ considered the fact, that the concrete receives shocks far better than brickwork with the numerous joints. -- Finally, reference is made to this, that the compound construction finds extensive use for fortifications just on account of its great ~~is~~ elasticity.

In the last great earthquakes in San Francisco and Messina, brick buildings collapsed like houses of cards, while reinforced concrete structures held firm. ² One here has to do with an extraordinary stiffness of the whole, in consequence of the monolithic construction of all parts, -- floors, girders, supports -- there can result no sudden destruction. The floors are constructed with great rigidity, and no portion acts separately without affecting the other parts. With such great vibrations as occur in earthquakes, they would first show cracks and flaking of the plastering. But the structure will not fall at once; in spite of cracks produced, it preserves at least ~~it~~ for a time its cohesion, so that the occupants have time suff-

panels in the most rapid and simple manner. Even the removal of stairway walls must for the reasons mentioned meet with no special difficulties. A later insertion of beams or alteration of the floors or girders is perhaps not well possible; at least the cost of such changes will be disproportionately high. The later additions of transmissions or installations for light and power present no difficulties, on the contrary. Conceivable and costly is the subsequent insertion of an elevator, the transfer of a stairway, etc., since in such cases -- by cutting openings -- the continuity of the floor is destroyed. ¹ Naturally difficult is also the later removal of supports; yet it is to be considered, that already in the designing was this in view from the beginning, to arrange the fewest supports possible, also the widest spans, in order thereby to produce a truly manifold possibility of alterations in the use of the interior and in its subdivision. But although the removal of such supports should become necessary, then one opposed the scarcely greater differences -- aside from the matter of cost, as in pure steel construction -- later structural alterations are always bad, and it is finally no disadvantage, if one structural material opposes greater difficulties to the difference considered than another.

Note 1. p. 19. The city building department of Breslau in 1911 issued a supplementary order, in which is stated among other things; "Indispensable removal work is either to be done with a previous understanding with the firm, that erected the structure in question, or be executed with that firm itself." (See Section V A).

^{difficulties}
Likewise are considerable differences in the way of the removal of reinforced concrete structures. As a rule, one is compelled to use rams, lever shears, steel saws (better is the oxyhydrogen blowpipe for cutting through the steel), compressed air chisels, oxygen blowpipes, explosives, etc. For other structures to be torn down, the gain from the old materials as a rule exceeds the cost of the necessary destruction. Stone and brick can be used again, even steel suffers a relatively small loss and cannot fall below the value of scrap, so that it can always be sold again at about $1/4$ its cost. Wood can at least be sold again for fuel. On the contrary exists the fact, that the destruction of a reinforced concrete structure not only br-

brings in nothing, but also entails considerable cost. And as for rebuilding on a site, on which was a structure of concrete or of reinforced concrete, then under some circumstances even a certain decrease in the value of the ground may be stated, particularly when great masses are to be broken up. Loading is difficult and the further reduction is costly. Besides the fragments are not well usable for filling purposes. If these are also drawbacks, then one must consider well, that in general it is opposed to the feelings of a specialist to so shape his building, beforehand ^{and} to select his materials, that a later removal can easily occur. ¹

Note 1. p. 22. See also the essay, "Destruction Value of Concrete Structures." T. & S. 1910. p. 604; also S. & S. 1913, p. 225, 227, 1914, p. 57; Cement, 1914, p. 189; also Schick, "Beton und Stahlbetonbau." 1914, H. Ernst & So. Berlin.

The objection is further made to reinforced concrete, that it is too heavy. But that cannot be such injury, especially for workshops, where one has to do with the effects of moving forces. The great mass is even the best means against the changes in form of the second order, and the additional stresses & produced thereby. Indeed only in special cases will one reject the use of reinforced concrete in consequence of its great weight, thus for example in hall structures on very bad soil, where by the use of a light wooden construction, heavy and costly artificial foundations are saved, or for roof trusses of dwellings, where a frequent subdivision of small attic is necessary, and a wooden roof with rafters presents by far the cheapest solution. But as soon as the attic are to be used for commercial purposes, as always the case in industrial buildings, reinforced concrete -- also on account of its fireproof nature -- again comes strongly into consideration.

The objection is further made to reinforced concrete, that in its use for floors it transmits sound to a particular extent. One must distinguish between transmission of sound through the air and that by the structural members set in vibration. But experience so far has taught, that the effects of resonance for reinforced concrete floors are not greater than for other floors. As the chief cause of the difficulty may be regarded the vibration of the hard and elastic concrete, which is fixed

to the walls. If one will not use any hollow floor,¹ then are recommended loose sand or fragments of pumice stone concrete fragments in a layer 3 to 4 cm thick, or even cork slabs, felt with linoleum, etc., as a means with a good effect, but naturally resulting in an increase of dead weight and of cost of the floor. Likewise these only reduce the transmission of sound vertically, but less so the vibrations of a continuous mass in a horizontal direction.² Cork layers are also inserted in the wall masses. (See Part II. 7 1st edition, p. 143).

Note 1. p. 21. For example Stugsart, Västmanl. Kersti, etc.; Gylling; Kersten. Part II, 7 1st edition.

Certainly it will be asserted, that communicating hollow spaces make the soil still worse, since the enclosed air produces a good transmission of sound, and the hollow spaces certainly form a good resonant floor. It would be suitable in some cases to fill the hollow spaces with light insulating material, for example with peat dust. The hollow spaces do not act directly to diminish the sound, but indirectly in consequence of the increase in the thickness of the floor.

Note 2. p. 21. Among others, see Beyer on "Experiments on the Construction of Floors and Walls free from Acoustic Objections". S & S. 1911. p. 181, 182. Also mentioned by Gullner in Betts of Concrete Construction. 1913. Heft 8.

and finally, the objection is still made to reinforced concrete, that its structural treatment requires a thorough knowledge of engineering science, and further a severe, conscientious, frequently unpleasant supervision by building officials, that one cannot detect the mistakes made in the reinforcement after the placing of the concrete, that in general the recalculation of an existing girder is impossible, if no drawings are at hand.

On formation of cracks, see Section V B as well as p. 30.

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resistance to 22 to 27 kil/cm², tested according to German standards. ¹

Note 1. p. 28. According to German standards, the tensile strength is now eliminated, whereby the requirements for crushing resistance of standard mortar was increased. After the introduction of the standard a considerable increase of the resistance to compression actually occurred.

Note 2. p. 28. The Laboratory of the Union of German Portland Cement makers in Karlsruhe supplies German standard sand at a price of 5 marks per 50 kil.

The German standard sand is obtained from a tertiary quartz bed of the blinite formation in the vicinity of Prottenwalden-O. The nearly white crude sand is washed and artificially dried by machine. The sifting of the sand is done by swinging sieves suspended like a pendulum. If one sieve is first taken out the coarse grains, then the fine by another. A sample from each day's work is subjected to a test of size of grains and purity in the Royal Testing Laboratory in Gross-Lichterfelde.

For supervising the size of grains sieve steels made of brass sheets 0.25 mm thick with round holes having diameters of 1.25 and 0.75 mm.

The standard sand found good after repeated control tests is sacked, each sack being closed by the lead seal of the Royal Testing Laboratory. (German standards).

Note 1. p. 29. The German standards require the following resistances to crushing:--

- a. After 7 days in water, at least 120 kil/cm².
- b. After 28 days in water, at least 200 kil/cm². (Formerly 160 kil/cm².)
- c. After combined hardening, at least 250 kil/cm².

Portland cement is essentially more permanent in volume than other cements, i.e., heat and cold as well as wet and dryness affect its volume in far less measure. Thorough experiments have indeed proved, that Portland cement in water experiences a slight expansion and in air a small shrinkage, and that these two phenomena appear more strongly at the beginning of the hardening, to finally entirely disappear in the course of time.

But as a rule, it has not been necessary to take into account such changes in volume on account of their insignificance. Yet if one desires to do so, then for great masses care may be tak-

taken to make thorough joints, then filling these with an elastic material, best asphalt. The leaner the mixture, the less will be the changes in volume, but so much less will be the strength. ² The permanence of the volume of the cement will also be increased by the fine grinding of the cement.

Note 1. p. 24. For a mortar of 1 cement + 3 sand the permanence is about 1 third as great as for a mortar without addition of sand.

Not unimportant in such purely natural changes of volume is the so-called "movement", a fact that can be frequently noticed with defective cements, and that is to be sought in a strong unnatural expansion of the already set cement. By hardening under water such changes in volume appear earlier than in air.

Movement may be caused by:--

- P.30 Too coarse grinding of the raw powder as well as of the cement.
- Imperfect and unequal mixing of the raw powder.
- Too high percentage of lime.
- Imperfect burning.

High percentage of materials affecting the volume, like asbestos, ¹ combinations of sulphur, etc.

Strongly moving cements are indeed able to crush or spring external walls. With good and conscientiously made Portland cement, such movement is entirely prevented.

Note 1. p. 31. Movement in consequence of too high percentage of magnesia only occurs late after completed setting, but is then particularly dangerous.

The movement of a cement is recognized by the appearance of the so-called "movement" cracks, that are to be referred to bad preparation and differ from the shrinkage cracks first appearing after complete setting. The movement cracks generally start from the edges of the structural members, while the shrinkage cracks occur more in the interior. Not to be confused with these movement cracks are the "hair" cracks, which appear in older cement blocks in the open air, and are caused by the frequent alternation of cold and dryness. Particularly in plastering in the open air, hair cracks are hard to prevent, especially with too rich a mixture. It is also a fact, that concrete blocks of wet concrete (see p. 41) show greater inclination to hair cracks, than those made with a dryer mixture.

A good means for preventing such an appearance is the addition

arrangement of blind joints in the plastering, which divide the entire surface into smaller parts. It is also advisable to keep the plastered surfaces wet during the setting period by means of wet clothes and spraying. One likewise protects the fresh coatings from sun and wind, since otherwise the moisture is too soon taken from the mortar. ² With rough concrete surfaces the cracks do not appear as quickly as on smooth surfaces. Hair cracks are not injurious, so long as the materials producing rust (Oxygen, carbonic acid and dampness) cannot penetrate to the steel; that occurs only when the limit of elasticity of the steel is exceeded.

Note 2. p. 30. Also see Section IV. On tension cracks (extension in structural members in consequence of the effect of external forces, see Section VIII.

"Shrinkage cracks" already occur in the setting period (especially with too fine grinding) by too rapid drying in a current of air, by the heat of the sun, or by frequent alternation of dryness and dampness, but are entirely unimportant. Hair and shrinkage cracks may be best avoided by addition of a sand (at least 1:1). Sand makes the cement weatherproof, already with the mentioned proportions of 1:1, a characteristic not possessed by pure cement without addition of sand. Thus to reinforced concrete is not applied a coating of neat cement, since shrinkage cracks are then unavoidable.

3. Testing Portland Cement.

In regard to testing cement for its mechanical characteristics (standard testing), reference is made to the corresponding regulations of the different officials and countries.

It may only be mentioned here, that according to the German standards a test of tensile strength is not required, that as the properly determinative standard test is taken the crushing resistance of cubes after 1 day in air and 6 days in water, 21 days for hardening in the air. As preliminary tests are the testing of cubes after 1 day in air and 6 days in water. Tests of the neat cement are no longer necessary according to German standards.

In the German standards is prescribed testing by parts by weight. But at the building site the mixing is by volume, which has the result, that cements with smaller volume weights exhibit strengths in testing, that can seldom be attained in

mixing by volumes.

4. Steel Portland Cement.

Steel Portland cement is obtained by grinding melted ^{granulated} slag from smelting furnaces ¹ with Portland cement clinkers, burnt from smelting furnace slag and limestone. The color of steel Portland cement generally varies from light gray to grayish black. The specific gravity in airdried condition is 2.90 to 3.13. The fineness of grinding is the same as for Portland cement, and also the setting and hardening periods. ¹ Yet it must be recommended to make the mortar with rather more water than for Portland cement. The changes in volume are likewise greater than for Portland cement; in experiments the steel Portland cement even behaves better than Portland cement in regard to expansion and cracks. Also a movement of steel Portland cement is not to be feared. Once set, it is as good as unaffected by frost, heat, bog, sea and salt water. Steel Portland cement bears storage just as well as Portland cement. The first judgment, that with steel Portland cement, on account of its somewhat smaller weight per volume, more cement must be used than with Portland cement, that the work is dearer, has proved not always correct. Also in regard to the strength to be obtained, steel Portland cement is not inferior to Portland cement. Experiments of Royal Materials Testing Laboratory at Gross-Lichtenfelde. 1912. Heft. 8. ²

Note 1. p. 31. Smelting slag sand is a clay silicate poor in lime, that is granulated by running the slag obtained from smelting furnaces into cold water (sharp "foundry sand"), or by changing it into dust by steam or air jets (the slag appears as small enameled spheres), and in this way it acquires hydraulic properties, which the slag did not possess previously. Drying then follows by a preliminary heating in rotating drums, that are either heated by smelting furnace gases or by a coal fire.

Note 1. p. 32. That the slag sand has hydraulic properties and takes a material part in the setting and hardening processes, has long been contested. The slag sand is commonly termed ^{more by} a diluting material of Portland cement, in the place of which well ground sand could be used. Yet by recent researches the incorrectness of such assumptions has been proved.

Note 2. p. 32. See Zeits. ^{of} Union of Architects and Engine-

3. Satisfactory air hardening of the cement is to be shown by repeated tests.

- P. 34 According to experiments at the Materials Testing Laboratory at Gross-Lichterfelde, of Wayss & Freytag Co and other firms, protection from rust of reinforcement in steel Portland cement exists in equal degree as for Portland cement. According to Contribs. of the German Commission (1913, Heft 22), the behavior of rusty steel bars in steel Portland cement mortar is apparently somewhat more fortunate than in Portland cement. The experimental mass was for 5 years exposed to the action of air, fresh water, sea and bog water.

Steel Portland cement has already found frequent use. The "Union of German Steel Portland Cement works" produces for the trade annually over 233,000 tonnes of cement. This Union binds its members to make the cement exactly according to the description.

5. Slag (Smelting Furnace) Cements.¹

Note 1. p. 34. The name of "Smelting Furnace Cement" first appeared in recent times.

Slag Cements are products, that are made from granulated slag from smelting furnaces -- and as a rule at least -- with the addition of Portland cement or hydrate of lime.² Such cements can sometimes satisfy the standards, but often entirely fail to do so in practical use with other additions. They are not to be confused with steel Portland cements, which in opposition to the slag cements, consist in the greatest part of Portland cement. But in slag cement the slag content predominates.

Note 2. p. 34. All slags have the peculiarity of setting not at all or very little. The addition of materials containing lime is necessary to make their hardening possible. (Lime sulphate or Portland cement clinker, which always contains an excess of lime) (See Cement. 1914. p. 96).

B. Materials added for ~~Cement~~ ^{Concrete}.

By frequently shoveling over Portland cement with sand and gravel or crushed stone (aggregate) with the addition of water at the same time, an intimate mixture is produced, that is termed concrete.³ The size of grains of the aggregate must be determined by the use of sieves with different sizes of mesh.¹ The density of the material depends on the combination of the grains. Sands and gravels exhibit such great diversities, that

According to the General Regulations are to be understood:--
as sand; pit, river, sea, crushed sand, etc., up to grains 7 mm in diameter; slag sand (granulated melting slag of suitable composition, pumice sand and gravel, may also be used.

As gravel; gravel, gravel stone, pebbles of 7 mm up to 7 cm in greatest diameter.

P. 36 As gravel sand; the natural mixture of sand and gravel, as found in trenches ~~and~~ at bottom of waters.

As crushed stone; crushed stone between 7 and 25 mm.

As broken stone; stone broken by hand or machine between 25 and 70 mm in greatest diameter.

According to H. Burcaartz, Gross-lichterfelde West, for preparation of mortar and concrete the following additions are considered.

1. Sand (heap of finest and coarsest grains up to 5 mm (7 mm).
 - a. Natural sand (pit, river, sea, dune, volcanic, gravel sand, as well as crushed sand).
 - b. Artificial sand (for example slag sand).
2. Crushed stone, gravel, gravel sand, chips (partly sand, partly more grains of pea size up to about 25 mm).
3. Spalla (aggregate, chips, crushed stone, with grains over 25 mm).

- a. Natural stone (pebbles, granite, basalt, limestones, etc).
 - b. Artificial stone (bricks, slag, crushed concrete, etc).
4. Residues of burned coal (ashes, coke, slack, cinders).

For deciding on the goodness of the aggregate, the following characteristics come into consideration, according to Burcaartz.

1. Nature of grains.
 - a. Size of grains.
 - b. Form of grains.
 - c. Surface character of grains.
2. Density of the pile.
 - a. Weight of volume (poured, filled, shaken down).
 - b. Specific gravity.
 - c. Degree of density.
 - d. Lack of density (volume of interstices).
3. Combination of grains. (Condition of variations in size of grains in the pile).
4. Chemical nature.
 - a. Content in parts of loam (sand or clay).

h. Content in parts of other injurious substances (humus, peat, coal, sulphur, burnt lime).

R37 c. Content in excluded silicates (volcanic sand, pumice sand, slag sand).

5. Strength and physical characteristics.

a. Resistance to crushing.

b. Absorption of water (porosity).

c. Permanence in frost (weather).

d. Resistance to fire.

Addition of Sand and Water.

For the intimate cementing together of the coarser aggregate is used a mortar of Portland cement and sand grains of mixed sizes from 5 to 7 mm, thus as large as peas. This does not need to be a pure quartz sand, but may also contain hard limestone parts. In general we distinguish between natural and artificial sand (slag sand). To natural sand belongs not only pit, river, sea and dune sand, but likewise sand produced from natural stone by foot or ball mills, like granite sand, basalt sand, etc., as well as pumice sand. Like the stone, it must be free from adhering earthy mixtures, from residues of coal¹ and plants; sand must show no earthy or clayey parts,² which adhere firmly to the grains. Plant mixtures come from the water added and injure the mortar. Likewise care is to be taken, that no sulphur grains are to be found in the sand, which with a lens can easily be recognized by their shining golden yellow gleam.³ Sea sand can only be used when the salt is removed by washing. Slag sand obtained by granulating the slag from smelting furnaces, mostly reduces the strength of the mortar in considerable measure, wherefore its use in general is not advisable. (also see p. 45).¹ Pumice sand as a rule affords sufficient strength.²

Note 1. p. 27. Coal in sand, especially when too large (anthracite lumps) can produce movement; also great care is required for watertight plastering.

Note 2. p. 37. The existence of a loamy mixture can be recognized by rubbing the sand between the hands until a certain amount occurs, and a marked loamy odor is perceptible. -- If the loam (in small measure) is finely distributed in the sand, it is not injurious; only it must not adhere to the grains.

Note 3. p. 37. One can judge of the goodness of any sort of

sand easily quickly by the following test:-- take a glass of clean water and shake some of the sand into it. If the water remains clear and is not turbid, the sand can be used at once. Otherwise a preliminary washing of the sand cannot be omitted. For extensive works is chiefly employed a machine for washing the sand to save cost and time.

Note 1. p. 38. But also see Report No. 17 of the Smelting & Furnace Commission of Union of German Iron Works. (Report on question of slag sandstone). Stahl & Eisen, Dec. 9, 1911.

Note 2. p. 38. Pumice sand is a volcanic slag, hardened under the influence of water, obtained in Germany in the Rhine and adjacent regions. (See p. 15).

Washing the sand is not always available. Men are very frequently deceived in its appearance, and believe that a sand with little clayey and earthy particles must show less resistance than the pure and apparently very good sand. But experience often shows the contrary; a very sharp sand,³ apparently entirely pure, has already shown itself as unusable in many cases. For those clayey particles, which may exist up to 10 p.c. (not in lumps but uniformly distributed), only affect favorably the density of the concrete, so far as they do not adhere to the sand grains; then they even frequently give it a firmer adhesion and a greater resistance. Therefore one should not regard all apparently dirty sand as impossible for making concrete. It is best to have the sand tested for resistance before using it. Such tests are even appropriate, since even the goodness of any sort of sand cannot be determined merely by touch and sight.

Note 3. p. 38. By "sharp" sand is understood a pure sand with angular and not too small grains, which produce a rattling noise when compressed in the hand. Contrasted with this is the "soft" sand containing loam with rounder and very small grains.

The more angular the grains, the rougher the surface, the firmer the original stone, from which the sand was derived, and also the stronger the concrete.

What finally concerns the size of the grains of the sand is, that a sand with mixed grains (up to 7 mm size) with a high weight per litre is best. Too fine a sand⁴ requires the addition of much water, and produces a mortar with less strength. While too coarse a sand would demand too much cement. From

Figs. 3, 4, it may be readily seen, that for cementing together the grains of sand, more cement is necessary in the first case than in the second; for the voids are then greatest if the sand consists of grains of equal size. But to save unnecessary cost of freight, in most cases -- without more thoroughly testing its quality -- will be preferred that sand located nearest the place of its use.

Note 4. p. 38. According to the Swiss Regulations, fine sand that will pass through a sieve with meshes 0.5 mm in size, should not exceed 10 p.c. The Austrian Regulations require a larger sand, 35 p.c. being left on a sieve with 300 meshes per cm² with more 0.1 mm diameter.

The basis assumed for the resistance of sand (according to Gary) must be 150 to 200 kil/cm² resistance after hardening for 28 days, mixed in proportion of 1:3. (450 kil cement per m³ of mortar).

The water required for making the mortar must first of all be pure, and thus free from impurities, with a proper temperature when used. Water too warm shortens the period of setting and too cold water lengthens it. Swamp water¹ and sea water are unsuitable for making mortar, as well as water containing gypsum and magnesia. Furthermore bog water with humus, peat fibres, or with oily spots on its surface is to be avoided, as well as all water from factories containing acids and salts. Most suitable are rain and spring waters, as well as water from rivers and lakes. As for the limit of the quantity of water added, then one is referred to experiments for its determination. It depends both on the purpose of the building, as well as on the more or less porous nature of the building materials used. (Very porous aggregates are properly to be saturated with water before making the concrete). Likewise the weather and air temperature then prevailing are to be considered; in rainy weather and cold days the addition of water must be less, but on the contrary be greater during great heat. With insufficient water the cement sets imperfectly; but with too much water on the other hand there result small crevices filled with water, which later appears on the external surface. But determining the best choice of the volume of water are the necessary tempering and the duration of the hardening of the concrete. The amount of water added is likewise to be so determined, that the

elasticity of the concrete only results after long and heavy tamping, indeed with the observation of a slight moisture on the upper surface, the so-called "sweating." To test the degree of moisture of the concrete-mortar, one takes a little concrete in the hand, pressing it into a ball with the fingers. No softer must squeeze out between the fingers, but it must show water on the surface, the concrete mass retaining its form after opening the hand. Too great an addition of water injures the strength of the concrete; it would be badly tamped, since the mass would be too elastic and would soften. Tamping would no longer increase the density. However it must be admitted, that a rather wet concrete takes its form better and is especially suitable for layers that have to receive the steel reinforcement. -- A rule for the practical man is rather too much than too little water.

Generally men distinguish between earth-damp, plastic (soft) concrete ¹ and cast concrete. The former comes in consideration only for tamped concrete structures of great dimensions, but not for reinforced concrete. It sweats only after long and heavy tamping, since it contains relatively little water. Disadvantageous in the use of this concrete is before all the fact, that in tamping constant supervision is required, and that only trained workmen can be employed, since insufficient tamping or an insufficient addition of water has the most injurious effect.

Note 1, p. 41. General Regulations give the following definition of earth-damp and soft concrete.

Earth-damp is a mass of concrete, that contains only so much water, that ^{only} after the completed tamping, the surface of the water begins to exude water or to yield under pressure of the finger.

Soft concrete is a mass with an addition of water so measured, that the edges of a deep depression on the already tamped mass of concrete remains a short time and but slowly disappears, so that a condensing effect of the tamping may still be expected.

The plastic (soft) concrete has a greater addition of water, has a greater addition of water, about 12 to 13 p.c. of the volume of the concrete. ² For reinforced concrete structures it is alone considered, at least in the tension zone of members subject to flexure; ³ the reinforcement of the floors are entirely enclosed and are not visible on the underside of the fl-

floor after the centering is removed. Finally, a smaller addition of water suffices for the supports, since there is required more a sufficient resistance to compression rather than the bond resistance between concrete and steel. Experiments of Bach¹ have shown, that earth-damp concrete after 23 days exhibits a greater strength than plastic concrete, but that in the course of time, the concrete last named rapidly increases in strength, and that after 100 days is scarcely inferior to earth-damp concrete. If one finally considers, that the placing and tamping -- in consequence of less tamping -- proceeds more rapidly and simply than with earth-damp concrete, then is one indeed justified in the decision, that the plastic concrete is entirely preferable to the other for the purposes of reinforced concrete construction. With earth-damp concrete no such unobjectionable enclosing of the steel reinforcement occurs as with the wet concrete.

Note 1. p. 42. Bach, "Contributions on making Concrete Blocks with different Additions of Water, as well as on the Compressible Strength and Elasticity of the same." Stuttgart. 1903. Also Bach, "Contributions on compressible Elasticity and Resistance of Concrete Blocks with varied Additions of Water".

Bach has found that the least quantity of water, that just sufficed to produce a perfectly tamped concrete, secures the maximum strength as well as the maximum bond resistance^{un} opposed to steel inserted in concrete and pulled or pushed out. (See Section VII).

Note 2. p. 41. "For very rich mortar mixtures with more cement than sand, is the addition of water to be considerably increased, up to about 30 p.c." Kurlenberg Regulations.

Note 3. p. 41. A sharp separation in a girder -- under soil and dry above -- would be wrong, since then the water separation of the two sorts of concrete would occur. A gradual gradation can be made with uniformly earth-damp concrete, the lower layer after being placed in the form being sprinkled with a watering pot and then tamped. The amount of water added must be gradually reduced upward. The chief advantage of this procedure is to be sought in the fact, that the machine has to deliver a uniform concrete.

If one employs (for walls, foundations and the like) a large great addition of water, so that the concrete ^{flows} down a board

inclined about 1:3 as a thick fluid mass, when one has to do with the so-called cast concrete. (Dumped or wet concrete). It is dumped from spouts on the centering and fills the small-est cavities. The work proceeds rapidly therefore, since all tamping is omitted. The concrete is particularly dense and with a smooth ^{centering} shows a dense and smooth surface, which does not further require plastering. For massive structural members, there may be inserted without hesitation well watered stones (boulders, split stones or the like). ² One is dependant on the exactness of the addition of water only in a slight degree. For a lean mixture the addition of water must be less than for a rich mixture; one can usually count on about 18 to 25 p.c. of the volume of the concrete. (Too great an addition may make the concrete too porous). Injurious to cast concrete is the fact, that a very strong and unyielding centering is necessary, which further must have sufficiently close joints, to prevent all leakage of cement. The concrete must be left a considerably longer on the centering than for tamped concrete. Finally, the assumption is entirely justified, that the compressible resistance is less than for tamped concrete; only in great structural members of considerable height does the compressing dead load weight of the mass above exert a favorable effect on the bottom layers. Cast concrete is employed to a great extent, particularly in America. ¹ (Also see Section IV3).

Note 2. p. 42. But in no cross section should be placed more than 40 p.c. of stones, distributed as uniformly as possible.

Note 1. p. 43. Further see Salts. of Union of German Architects and Engineers. 1912. p. 98, 177, 229; B. & T. 1914, p. 49. Ann. B. 1918. p. 440).

Stone and Gravel Aggregate.

In the choice of stone and sand, which determines the strength of the concrete, first of all are considered the kinds of materials found in the vicinity of the building site, so far as they are suitable for use. In any case, the stone required for making the concrete must at least exhibit a resistance to crushing equal to that of the hardened mortar. ² For this reason right hard stone, sharp edged, such as granite, gneiss, basalt, dolomite, grauwacke, hard limestones, etc., are especially recommended for making concrete, and the use of such stones always results in considerable increase in the strength of the concrete; ³ but a greater addition of stones than 20 to 30 p.c.,

or more than 40 p.c. is not to be recommended. The diversity in in the hardness of the crushed stone in itself has no substantial influence on the strength of the concrete. For example, a concrete of crushed basalt exhibits the same strength as concrete of crushed limestone, although basalt has about 4 times the breaking resistance of limestone. In case stone quarries do not exist in the vicinity of the structure, the transportation of crushed stone is certainly connected with a considerable cost for freight.

Note 2. p. 43. Austrian Regulations require, that the crushing resistance of the pure material shall be at least 300 kil/cm², and that the absorption of water shall not exceed 10 p.c. of the weight.

Note 3. p. 43. Basalt has an extreme hardness (crushing strength up to 3400 kil/cm², and the absorption of water is not more than 10 p.c. of the weight.

Note 4. p. 43. See B & Z. 1912, p. 226. (Concrete of crushed limestone). Limestone is readily ground and polished, but indeed may be destroyed by fire. It is cheaper than the eruptive stones, that in consequence of their hardness require considerably greater cost of labor.

Note 5. p. 43. On the fractured surfaces of concrete of broken stone, one frequently finds that the fracture goes through the middle of a stone, while in the gravel concrete, it always goes through the mortar; the bits of gravel mostly remain uninjured.

p. 44 In the choice of the sort of stone is also to be considered the fact, that the concrete later in great part is exposed to the effect of frost, wherefore porous stones, like crushed bricks (clinkers excepted) and soft sandstones, should be used little or not at all. Such porous stones will further produce a concrete of only small strength. Yet if -- for economical reasons -- the use of such stone occurs, then it is previously placed in water for a time, so that in the process of setting it may not absorb too much water from the cement mortar. Crushed stone is obtained from stone crushers.

Note 1. p. 44. The cost of such a machine amounts to 700 to 3000 marks, according to the power used (1 to 10 H.P.), and the production per hour of 0.5 to 5.0 tonnes, and with a mouth opening from 15 to 35 cm. The cost per m³ of crushed stone is

from 1.5 to 2.5 works according to hardness of the stone.

A very good and also cheap kind of stone is gravel, since in consequence of the most varied sizes of grains, it requires the smallest quantity of mortar. Fine grains increase the tensile strength, coarse grains the resistance to compression. Yet the strength of a gravel concrete does not attain an equally high degree as a concrete of crushed stone, particularly when the different grains are not angular but round. With angular grains the crevices between the grains are naturally less than for round grains. The tamping drives the angular grains like wedges into the crevices of the adjacent grains.² In general, pit gravel is to be preferred to river gravel, since the latter contains too little sand, and also the round grains are too smooth. Sea gravel is indeed very pure, but frequently contains salt, which later crystallizes out in the completed concrete. Gravel must be chiefly composed of quartz or strong silicious rocks, only exhibiting a small portion of sandy mixtures.

Note 2. p. 44. Many have indeed established, that round grains under some circumstances can give higher resistances to crushing than angular grains. Round grains are further also more easily mixed and tamped.

Finally, gravel may also occur in nature mixed with sand as the so-called "gravel sand". As a rule this presents the great advantage of being cheap and easily obtained. Its use is indeed forbidden by many officials, yet it may be used without risk, with approximately the right proportions of sand and gravel directly for the making of concrete. Necessarily the content of sand must be accurately determined by sifting, and be corrected by adding or removing sand. Admirable also is an addition of spalls, for example of basalt chips. If in this manner the use of a gravel sand be guaranteed, then is it to be permitted for the construction of reinforced concrete structures.¹ Only for statically highly stressed structural members (frame beams, pivot columns, etc.) are necessary certain considerations relating to its use.

Note 1. p. 45. See "Influence of the Size of Grains of Gravel Sand on Resistance of Concrete to Crushing." B. & E. 1911. p. 188, 193, 204. Also Arb. b. 1912. p. 154.

Pumice gravel sand (see p. 38) affords many advantages, is extraordinarily light, insulating for sound and heat, but is

dear, ² and in consequence of its porosity and small tensile resistance is very much inclined to shrinkage cracks. The permissible stress on a pumice concrete (with at least 300 kil cement/m³) can only be placed at about 15 to 20 kil/cm²; for higher values is necessary preliminary evidence of its resistance to breaking. Pumice gravel requires abundant addition of water in making the concrete; likewise the concrete must be kept quite wet until sufficiently hardened, and be protected from sunshine. With too little water content, the pumice concrete absorbs too much water from the cement and permits shrinkage effects. It must not be too strongly tamped, so that pumice grains are not crushed.

Note 2. p. 45. Pumice sand (grains 0 to 10 mm diameter) costs at the locality (Keweenaw-Is.) about 4 marks/m³. One can reckon for a car of 10 tonnes about 12 m³ pumice sand or 15 m³ of pumice gravel.

Pumice gravel is principally suitable for light roof construction and covering, where no alternating stresses and dynamic influences are to be feared. (For example 1 cement + 3 pumice sand + 3 quartz sand). The underside can be plastered with cement paste and twice coated with lime colors. The slabs (not ^{p. 46} I-beams) are made at least 6 cm thick and sufficient dividing steel rods and expansion joints are provided. It is not advisable to use pumice concrete only for the tension zone with gravel concrete for the compression zone (to lessen the weight of the covering), since a later separation of the two kinds of concrete is to be feared. For the same reason it is not well to make the supporting beams of gravel concrete, but the slabs -- for better insulation -- of pumice concrete, for example for covering attics. (Yet such construction has frequently been employed with good results).

The use of acid slag (smelting furnace and boiler slag) locomotive cinders, garbage cinders), ¹ as frequently common in America, requires very particular caution and preliminary testing of their usability. Slag concrete -- like pumice concrete -- has many advantages worthy of consideration, is cheap and light, protects well from heat, and on account of its porosity is a poorer conductor of heat and sound than any other concrete. The structure of the slag is spongy, its weight of volume is but small. Yet the strength of the slag concrete is mostly but

small; the slag may destroy the concrete under some circumstances, and in compound structures, especially with too lean a mixture, the reinforcement rusts in consequence of the contained sulphur. ² The slag must have an acid and not basic reaction, thus not redden blue litmus paper. It also easily inclines to shrinkage cracks and movements, especially with boiler and hearth cinders, and frequently has a shining smooth outer surface, to which the cement adheres badly.

Note 1. p. 18. Especially garbage cinders. See Contrbs. 1313, p. 167. Coal cinders are always more dangerous than smelter slag.

Note 2. p. 48. But the silicates of smelting furnaces slags are lighter than iron and swim on it as a protecting covering; they are let out by a special opening.

Especially dangerous is the use of fresh, unseasoned slag and cinders (coking, hearth and boiler cinders), that ~~may~~ contain unburned parts of the fuel and free sulphur. They afford opportunity for the formation of cracks and ~~can~~ ^{can} rapidly destroy the entire structure. Slag from fires must lie for at least 6 weeks in the open air before using, and during that time must be faithfully watered, so that the salts soluble in water may be leached out, and the lime grains be slaked at the same time. -- For purposes of concrete, the best suited is the basalt-like Thomas pig iron slag, since there is to be found a very small content of sulphur and phosphorus. -- The use of slag concrete naturally plays an important part for industrial regions; (Slag is a nearly worthless material); according to ~~the~~ previous experiments, aside from Thomas pig iron slag, one should proceed with very great caution in the use of slag for making concrete, and should allow it to be used only for concrete filling or subordinate parts of structures (walls etc.), but never for supporting floors, supports, etc. ¹

Note 1. p. A firm made test cubes at the building site from gravel concrete with the mixture 1:5; similar cubes were also made by the building officials on their part. By a later crushing test the blocks of the firm had an ultimate resistance of 300 to 400 kg/cm^2 , while on the contrary those of the building officials had only 50 to 60 kg/cm^2 . The resulting investigation showed, that the astonishing difference in the strength of the cubes had the following reason: the contractor had pre-

preserved his cubes in gravel during their hardening, while the building officials kept theirs in slag sand. This had evidently absorbed the water from the cubes, so that the water required for hardening was lacking in the cubes. -- In a Westphalian locality, under the market place was a concrete sewer, partly covered by sand, but partly by iron slag. When after some years the pavement was renewed, it was found that the concrete was so strongly corroded by the slag, that a greater restoration was necessary. Under the sand the same sewer was perfectly preserved without injury.

But on the other hand, official experiments have also shown, that a concrete with the mixture 1:5, made of smelting slag & similar to Rhine gravel, showed a higher strength than the same gravel concrete. (Z & B 1911, p. 352). -- One should also remember, that in Germany about 40 million tonnes of smelting & slag are produced annually, for which a new market could be very welcome. Besides the great slag mounds could be gradually removed to obtain room.

Further see Z & B. 1910, p. 177, 193; 1911, p. 352, besides the sources named on p. 32, Note 3.

A Commission of Officials of the Ministry of Public Works, of the Ministry for Commerce and Industry, and of the Ministry of Worship and Education, as well as delegates from the Union of German Smelters, from the German Concrete Union, from the Steel Works Society, and from the Engineers' Society, made a tour at the end of 1913 for visiting a large number of smelters. The Commission had in view an agreement on common measures and also an experimental programme, to produce a clear understanding of the usability of smelter slag in construction. The experiments are now proceeding.

As for the size of the grains, for gravel and crushed stone, this should not be over 5 to 6 cm in diameter, thus not exceeding the size of hen's eggs, but also other sizes may exist up to this extreme value. This is especially true for tamped concrete structures. Stones under 5 to 7 cm diameter are to be regarded as sand. The proportion of sand to crushed stone or gravel is to be determined by sifting tests. The sizes of grains for reinforced concrete are especially arranged according to the thickness and massiveness of the structure considered, and according to the size of the meshes of the reinforcement. For

solid and massive parts, where the steel rods lie at wide distances apart, larger pieces are allowed than for other thinner parts with closer meshes. In all cases one will always do better -- in the compression zone -- to make the size of the grains as large as possible (but for reinforced concrete at most 3 to 4 cm, since thereby a higher degree of strength may be obtained, it being naturally assumed, that the resistance of the grains is sufficient. In the tension zone is employed at most 15 to 25 mm, or in any case even 30 mm grains.

To make possible an intimate combination of the aggregate with the other materials, it is sometimes necessary to free it from adhering earthy particles shortly before use. Such washings also have the advantage, that the still wet stone in the proper making of the concrete absorbs less water from the cement mortar, than if it were used entirely dry. On the other hand, experiments have taught, that many gravel sands in washed condition exhibit a smaller resistance than in an unwashed condition. (Also see p. 38). Otherwise such washings are tolerably dear and troublesome, and can properly come into consideration only for the larger factories and the great building localities.

Fig. 5. shows the arrangement of a washer for crushed stone, as employed in the construction of a bridge of the local road Agomitz-Klaus of the K. A. State Railway. ¹ E is the delivered and F the completely washed crushed stone. The otherwise usual arrangement of pipes is replaced by 4 laborers, who shovel the material from E to D, D to C, C to B, and finally to F. The water flows in the opposite direction from A to E, so that the clean water meets the already cleaned crushed stone in the tank B.

Note 1. p. 48. See B & E. 1910, p. 15. -- In regard to other arrangements without machines, see E & B. 1911, p. 400, 429.

For more extensive work are also recommended special machines, the so-called gravel washers and sorting machines, which mostly consist of a rotating drum, and which work in the direction opposed to the stream, depositing the sifted materials in cars set underneath. ¹

Note 1. p. 48. Further see E & B. 1915, p. 352. Machines for daily product of 25 to 50 m³ require 7 to 8 H.P. for driving. To have sufficient force to wash the raw materials, the water enters the drum with a pressure of 1.5 to 2 atmospheres. Lock-

Lacking water supply pipes, may be employed a water reservoir located several metres above the machine.

Finally to be mentioned is still, that if necessary the quality of the stone, as well as of the other building materials, is to be determined by accurate tests,² and that on these tests the usability of the concrete mixture may depend to a great extent.

Note 2. p. 49. In the first line comes into consideration the official Materials Testing Laboratories in Gross-Bichterfelde (Berlin), Dresden, Munich and Stuttgart. A list of the official and private testing laboratories is to be found in Beton-Kalender, part I.

C. Steel Reinforcement.

Best suited for the purpose is a mild steel² with a resistance to tension and compression of about 4500 kil/cm². (See new rules for quality of May 15, 1913. Section VII.)

The use of cast or blister steel is not advisable for economical reasons, though the sections of the required reinforcing rods may be considerably reduced thereby. Then must also be opposed to the higher resistance of the steel a corresponding high resistance of the concrete -- whenever possible.¹ Also such steel does not directly permit cold bending of the end hooks. In all cases to avoid substitutions, only one sort of steel should be used at a building.

Note 1. p. 50. Steel of high cost (nickel steel) will less come into consideration for reinforced concrete structures.

Before their proper use, the reinforcing rods are to be cleaned with wire brushes from all paint, dirt and grease, as well as from rust, that loosely adheres. If the rust be firmly adherent to the steel, it has no injurious effect, but may even produce a stronger bond resistance, while dirt and grease diminish the latter. Many regulations require a previous coating of the steel with liquid cement (pure cement wash or mixed with sand 1:1), which is indeed very good, but not always necessary.²

Note 2. p. 50. Cleaning with acids, oil, lime, etoe, is to be avoided.

Professor Garner in Stettin has established, that the bond resistance is very unfavorably affected by frost, but the bad effect is again completely removed by coating the steel rods with neat cement. Other experiments have also shown, that ste-

steel previously coated with liquid cement has a greater bond in this concrete, than steel without such a coating. A previous coating is also recommended for the case, -- for example for foundation slabs -- if steel rods must be placed in leaner mixtures of concrete, but without having to fear later effects of rust. -- For ordinary cases however, one may omit a previous coating of the rods.

^{R. 57} The steel is best taken to the place of use in equal lengths if possible, (8 to 12 m), and with the fewest possible differences in sizes. Greater lengths make higher cost of freight, and too many slightly differing sizes of rods for the same building give opportunity for mistakes. For example, for beams and supports are taken 16, 20 and 30 mm rods, for slabs 10 and 13 mm rods with 5 mm for stirrups.

The steel must be spliced since (without considerable extra cost) it is not in the trade in the required lengths. Such splices can be made by screws on each end with a coupling over them -- the latter bored from a larger rod (?) if possible. ^{at} These splices are to be so arranged, that a section exist at most 2 splices. They are always to be placed in compression as in tension rods -- where the steel is stressed but little. The screws must be cleaned before joining and be coated with thin cement milk wherever possible. For thinner rods it suffices to lap the ends with hooks 20 to 30 cm and to wind them with wire. (Fig. 6).

A distinction is generally to be made between joints in compression and in tension rods. For compression rods already suffices a butt joint in a coupling (gas pipe) without screw threads, with a piece of lead between. Welded joints are less common; they are mostly not allowable for tension rods, because of the loss of strength in the rod, but are also dearer. Better, though no less expensive, is already the autogenic welding ¹ But ^{A. 58} most advantageous is it to avoid all splices if possible, especially at strongly stressed places (tension, buckling), or to lap the rods furnished with end hooks, and to wind them strongly with wire (about to the length of 30 diameters of the rod) at most to 50 cm.

Note 1. p. 51. Autogenic welding (with acetylene-oxygen flame). Is most employed for cast iron; cheap in use, portable and simple. This welding is termed "autogenic", since neither flux,

hammering or pressing together of the pieces to be welded is necessary for producing the junction. Injurious is the fact, that in autogenous welding a burning of the steel in the vicinity of the weld can scarcely be avoided.

Experiments of German Commission for Reinforced Concrete. § 14: Experiments on reinforced concrete beams for determining the resistance of ^{joints} ~~beams~~ in the steel reinforcing rods, have likewise shown, that with both end hooks as well as the winding with wire are to be recommended.

The cutting of the rods to definite lengths is done with the flat chisel, indeed either cold -- or hot for strong steel. But men generally employ special machines, the so-called steel shears, easily portable and driven by hand. ¹ According to size of steel will the rod be turned over or not. These rods are also cut with wire or plate shears, or by a bolt cutter. Larger rods (for reinforcing beams) are brought to the building in the correct lengths, if possible, in order to avoid the work of cutting and waste, in spite of a higher price. Wastes from thinner rods (reinforcement of slabs) can be more easily utilized. If for large rods (over 25 mm diameter) abrupt bends are necessary, then must the bars be first heated at the bends, then bent by hand (Fig. 7). Slight bends can be made cold on a mould by hand or with a screw press. In a timber are set iron pins, against which the steel is placed and bent. As a test the bend can be sketched on the drawing board in chalk and the rod be laid thereon. Points for bending are marked with chalk, never with a chisel. Fig. 7 shows a bending plate, into which are set ^{p 53} strong pins or bolts to serve as the form. ¹ The bending of the floor and beam rods is done in the workshop beforehand, if possible, and the steel in prepared condition is delivered by rail or wagon.

Note 1. p. 52. Preferable are such steel cutters, where the rod is not passed in from the side frequently for several yards, but where it can be directly placed in the cutter at the desired point.

Note 1. p. 53. On end hooks and bends, also see Section XII. It should be noted, that a small percentage of silicon or phosphorus may be unpleasant for bending, although it does not unfavorably affect the strength.

The bending of the rods requires much payment for labor. In-

Therefore tools have been made, that make it possible to bend cold rods up to 4 cm diameter without much labor. These tools allow the preparation of the rods on the building ~~site~~ itself, so that no mistakes or losses can occur. For example, Fig. 8 shows a rod bender for edges of vaults by Pfeiffer and Ludwig, Mannheim, with which ~~cold~~^{round} rods 36 mm diameter can be bent cold.

The former generally careful and usual preparation of the steel in the smith's shop is not only lost time, but also requires considerable cost of transportation, that will be avoided by the use of bending appliances. The rods are taken in their usual trade lengths to the building site, and as needed are quickly cut and bent. The foreman can take dimensions at the place and then have the steel prepared at once under his oversight and placed. The small scraps amount to about 10 p.c., and can be used for single and double stirrups, or for larger diameters (more than 5 to 7 mm) can be used for anchors, distributing and supporting rods, or also for reinforcement of vaults, footings, etc. Possible alterations can be made at once at the site.

p. 54 As for the section of the steel, the round rod is the most common. ¹ Round rods favor the escape of air and the tamping of the concrete; also they have no sharp corners, that cut into the concrete. Any later alteration in the structure can be readily considered, which is not the case with rolled shapes. They are also to be obtained at all times and at any steel store. It is indeed injurious, that the round section -- for a given sectional area shows the smallest coefficient for bond stress (see Section VII). Flats and squares are also employed; that are sometimes twisted about their axes to increase their bond resistance (Fig. 9), indeed after previously heating. (Twisting cold produces unfavorable stresses in the steel).

Other sections are the T, L, I and S shapes. Further triangular and square rods with concave sides sometimes come into use. ²

Note 1. p. 54. But it is advisable to test the sections on delivery, since there has occurred a difference up to 10 p.c. between the ordered and delivered steel.

Note 2. p. 54. All these special bars (Hansons, Thatcher, Johnson bars) are dearer than round rods; they are more common in America. Experiments have shown, that the varying sections

In consequence of the projections and hollows, etc., may crack the enclosing concrete.

To connect intersecting bars (bearing and distributing rods), one takes $3/4$ to 1 mm annealed wire, the so-called winding wire. This is wound about the rods, held with pliers, twisted and fastened. For the Stirrups (see Section XIII) are used round rods 5 to 6 mm in diameter and sometimes thin flats.

On "spirally wound concrete", where spiral or annular bands are employed, see Section XVII.

Also recently various special bars are in use, that afford certain advantages over round rods. They will first of all make the superintendence independent of the usual defects in directing and setting the reinforcement, and render possible a simple and certain checking by the superintendent. The preparation of the steel at the building will be limited to a minimum; likewise the least dependence will be placed on the skill of the workmen entrusted with the execution.

Rolled shapes, such as T-beams and railway rails, are stiff reinforcements which simplify the centering and make possible a more rapid composition of the structure.

Built shapes (Pohlmann system) are rail sections with perforated webs. The connection is made by ring-shaped stirrups or slings as shown in Fig. 10, which fasten in the holes and are fixed by iron wedges in the larger buildings. The lower flange is considerably larger than the upper flange. ¹

Note 1. p. 55. See Kerster, Part II. 7th edition. p. 42.

In the construction of the so-called Lolat-beam, the starting point was, that the I-beam when employed in concrete construction almost always caused a loss of material. The Lolat-beam (a lattice beam) has a removable top flange, which receives the compressible stresses before the hardening of the concrete. After the setting the upper flange is removed, and its statical purpose is transferred to the concrete. -- Also see Gontzke, 1911, p. 104.

The rivetless lattice girder (Fig. 11) from the "Manufactory of the Rivetless Lattice Girder Co.," Düsseldorf, is pressed from a flat plate with double cuts. It consists of two bottom chords at the same height and an upper chord, solidly connected by inclined bars on opposite sides and directions. Such a lattice beam affords a faultless connection of the compression and

tension zones of the concrete; it is perfectly anchored in the concrete, and joins without rivets all tension and compression reinforcements, stirrups and bent rods. The firmly enclosed oblique struts receive perfectly all shears and bond stresses, and further the side bars ensure an increase of the bond resistance and that to slip. The lattice beam is delivered complete; it requires no preparation at the building. In consequence of its stiffness the beam can itself support a part of the centering, therefore reducing the number of supports. The beam is placed on the market in various sections. ¹

1. p. 56. See Contrbe. 1913. p. 59.

Kahn bars are angular bars, whose two opposite angles have wings. (Fig. 12). Toward the end of the bar, these wings are cut loose, bent upward and utilized as stirrups to receive the shears. (Fig. 13). Thus they are firmly connected with the main bar; any slip of the bar in the concrete is prevented. The Kahn bars come to the building ready to set, indeed in different sections. By the Berlin building office has been allowed for Kahn bars a tensile stress of 1200 kil/cm² since 1912; calculated proof of the magnitude of the bond stress is not required here. ²

Note 2. p. 56. Kahn bars are rolled with 4 sectional areas of 2.55, 3.10, 3.95 and 12.75 cm². See Kersten. Part II. 7 th edition. p. 41.

Likewise the use of expanded metal as reinforcement for reinforced concrete structures remains to be mentioned. This is made from a steel plate, which is slit by a machine and then stretched in a direction perpendicular to the slits to form a lattice with lozenge-shaped meshes. ³

Note 3. p. 56. Expanded metal (Fig. 14) comes in consideration especially for simple structural members (walls, foundation slabs, freely supported floor slabs, etc.). The fear that under tensile stress no sufficient resistance to deflection exists, has been disproved by experiments, or that a lengthening of the network was possible; the concrete tamped between the meshes hinders any change of form. The laying of such a network proceeds rapidly, yet the right structural lengths do not always exist. Likewise the reinforcement cannot be so adapted to the course of the line of moments, as is easily possible for round rods. -- See Kersten. Part II. 7 th edition. p. 40.

The system of "Schroiff Steel Concrete" (Bielefeld) uses thinner but strong steel wires. The advantages of this system are often only of a quite conditional nature. ¹

Note 1. p. 57. Also see the American constructions with wire netting reinforcement. B & E. 1909. p. 138; 1914, p. 91.

For supports the use of cast iron may finally come in question. ("Spirally banded cast iron". See Section VI. 7.

SECTION III. MAKING AND USE OF CONCRETE.

A. Hand and Machine Mixing.

After delivery of the Portland cement in the original packages begins the proper mixing of the concrete, indeed as a rule by units of weight. The Prussian Regulations prescribe the following:--

"The concrete shall be mixed by units of weight; as a unit shall be taken the sack = 57 kil.² or the barrel = 170 kil of cement. The additions may either be weighed or measured in vessels, whose volume is determined previously, so that their weight corresponds to the required mixing proportions.

Note 2, p. 57. In south and west Germany sacks of 50 kil (= 46 litres) are in use. A sack of 57 kil corresponds to 1 1/3 cask (= 41 litres).

The mixing of the concrete must follow in such manner, that the mass of the different components always corresponds exactly to the required mixing proportions, and can easily be measured. In the use of measuring vessels the filling to be done so as to produce the best possible uniform density of the contents always in the same way."

The requirement of mixing by units of weight³ is therefore prescribed, since the density of the contents depends on the manner of shaking, and a uniform mixture cannot be produced otherwise. At this time cement strongly shaken in a vessel of definite volume does not show the same weight as loosely poured cement. Damp gravel and sand weigh more than if dry. Yet in practice men for economical reasons almost exclusively use wooden measures,¹ but where care is always to be taken, that the manner of filling the measure remains always the same, so that it is either always shaken down or loosely poured in. As a rule are employed wooden boxes without bottoms, consisting of 4 boards with handles for raising. They are placed on the mixing platform, filled with sand or gravel and then raised. Usually suffice the wheelbarrows, that are used for bringing the materials. If volumes by weight are calculated, then 1 c³ of Portland cement is taken at 1400 kil.² Average weights for gravel and sand are to be obtained by several experiments. For small hand mixing are necessary; two measuring frames (1 for sand and 1 for crushed stone), 2 wheelbarrows, 1 shovel, 1 rake, 1 water cask, 1 watering pot with hose, and 1 sieve for sand

with 7 shovels.

Note 3, p. 57. Mixing by units of weight may also produce a satisfactory result, since one is dependent on the dampness of the air. In Switzerland and in France (also frequently in Germany), the mixing proportions are given in kil for the cement and m^3 for the sand, gravel, crushed stone, which certainly makes the oversight easier. (Also see p. 52 to 70).

Note 1, p. 58. Sheet metal measures are not to be recommended, since by careless handling, they are too easily bent.

Note 2, p. 58. But see p. 25, Note 1.

The mixing itself may then follow in two ways; by hand or by machine. Mixing by hand is done on a firm and plane platform or mixing bed of joined planks, termed "mixing bed, mortar bed, mixing platform or bench", that is to be built in the immediate vicinity of the building site. To hold the planks 5 cm thick serve cross cleats, placed at distances of 50 to 70 cm. The top of the platform (about 10 to 12 m^2) must be smooth, for the shovels to slip easily -- in the direction of the joints -- and to prevent all catching.

Mixing by hand -- assuming two or three men -- may proceed in the following manner (Figs. 15 a to g).

a. First a definite quantity of sand is spread at uniform depth, then a corresponding quantity of cement is placed on it in uniform thickness.

b, c. Dry mixing:-- the men A and B shovel the sand and cement, beginning at their feet, over to the other side of the platform. By shaking the shovels the materials fall mixed on the floor. In this manner the dry mass is shoveled back and forth 2 or 3 times, until no pure cement is visible, and the whole -- uniformly spread -- has acquired a uniform gray color. (A third man can help with the rake in the mixing).

d, e. Then the stone measure is placed on the spread mass, the crushed stone placed in it (often wetted), the measure is again raised, and the crushed stone is uniformly spread.

1. p. 59. By the use of gravel sand the work is naturally simplified somewhat. Then suffices twice mixed dry and three times wet, without specially measuring the gravel.

f. Wet mixing:-- Now follows another mixing 3 to 4 times by shoveling back and forth (as for dry mixing), sprinkling at the same time ¹ by the man C, with strong turning and mixing

at the same time. The whole must neither show streaks nor dry spots.

Note 1. p. 60. The sprinkling also frequently occurs before shoveling over, or before the addition of the crushed stone. -- The Hamburg building officials, for example, prescribe, that "the water is first to be added to the dry mixture, only when the different components have been mixed together." The Kurtenberg Regulations (1912) require the same; they demand mixing three times dry and four times wet. -- But many workmen first make the mortar (with the addition of water), then adding the crushed stone. (See Note 1 on p. 63).

g. The mass is brought into a heap and is ready for use.

In the case described, 3 men are employed in hand mixing. [†] Two men would suffice, when the sprinkling already occurs before the shoveling over the crushed stone. The work of the third man C -- raking and sprinkling -- would then be done by A or B. If 4 men are at command, then larger mixtures are made on a correspondingly larger mixing platform. The men A, B, C, D are then best placed as in Fig. 16. The mixing is divided in the work; A and B shovel one half to one side, C and D, the other half to the other side. One or two men -- in rapid work -- can finally take the raking and sprinkling, as well as bringing in the materials.

Note 2. p. 60. A group of 12 laborers can in one day mix in this manner 15 to 20 m³ of concrete.

Naturally all washing while sprinkling is to be avoided, and an entirely uniform distribution of the water is intended by constant thorough shoveling. The entire process of mixing must be completed rapidly and without pauses, since after watering the setting of the cement begins at once. Therefore the quantities mixed are to be so measured, that they may also be prepared in a brief time. In warm and dry weather the concrete mass must not remain more than an hour, or in cool or wet weather more than two hours before using. Concrete not used at once is to be protected from the influence of weather, like sun, wind, heavy rains, and to be shoveled over again before using. After each mixing, the platform is to be cleaned off again. ¹

Note 1. p. 61. Further see B & E. 1911. p. 487.

On the transfer of the finished concrete to the place of use and the tamping of it, see Section IV, 3.

Mixing with machines is practically done the same as by hand. For large works mixing by beating is decidedly preferable, since a more uniform mass is obtained, independently of the skill of the workmen. ¹ The work of the machine is also substantially more reliable and is simply a necessity for large works, since thereby can be saved $1/2$ to $2/3$ of the cost otherwise. Great machines of 10 to 15 HP are able per day of 10 hours to produce up to 400 m^3 of mixed concrete, thereby considerably shortening the building period. (Filling the mixing drum 30 to 40 times per hour, which holds 0.1 to 1.2 m^3). One distinguishes between fixed and portable mixing machines, that are partly equipped with automatic filling and discharging apparatus. ³ Their use is mostly very simple, both for mixing as well as for filling and discharging, requiring but little labor, as a rule only 1 man. The materials are put in with shovels or wheelbarrows, mixed dry and finally mixed wetted. As a rule the wet mass is also kneaded for a time, finally to be taken by cars or wheelbarrows to the place where used. Most frequently driven by a portable steam engine set in a place protected from wind and weather, easily accessible for water and coal, and which may also drive other building machines, like hand saws, washing machines, pumps, etc., as well as to hoist the concrete, to the upper stories, and also for elevating stone, timber etc. The engine in this way takes all the work, that can be done by a machine. ¹ For smaller work suffices an electric motor ² or better a benzine motor, which is mostly cheapest in use, (using per day of 10 hours and 50 m^3 of concrete about 10 to 15 kil, or for 1 m^3 of concrete about 0.2 to 0.5 kil of benzine, and able to run promptly at any time.) ³ For attending the mixing machine, supplying materials and discharging concrete, one man generally suffices. (7). Least time for mixing is about 3 minutes (30 to 40 revolutions).

Note 1.p.82. The Austrian Regulations of 1908 require an increase of about 5 p.c. of cement for hand mixing.

Note p.81. The use of special measures for materials used is generally omitted. For the usual batch of at least 150 litres suffice wheelbarrows or dumping cars with known capacity, and the like. For the mixing proportions to be employed is taken a certain number of barrows or cars; the cement is measured in a cast made for the purpose.

Note 1.p.82. Advantages of portable engines for small at-

sites:-- Special constant attention; maintaining steam during pauses in the work. In general a portable steam engine with the use of less than 15 H.P. is seldom advantageous.

Note 2.p.62. An electric motor is entirely safer in use and requires the least attention. Disadvantages are often the quite important cost for the arrangement of the electric wiring. Also disadvantageous is the fact, that the electric currents at command frequently vary much in kind and voltage at the different building sites. One can count for 1 m³ of mixed concrete about 80 p.c. of the price of current for the cost, thus about 15 to 20 H.P.

Note 3.p.62. 100 kil benzine costs about 35 to 45 marks.

Note 4.p.62. See B & E. 1912. p. 414.

The machines deliver concrete with any preferred degree of moisture. The different mixtures can be furnished in exactly determined quantities and show the most perfect uniformity. The strength of machine concrete (compressile or tensile resistance) is quite great, and in consequence of the thorough kneading of the mixture is often 20 to 40 p.c. higher than the strength of concrete obtained by hand mixing. ¹ For machine work under some circumstances a leaner mixture can be used, which is able to present equal strength as a richer mixture by hand.

Note 1. p. 63. To obtain better values for strength, it is advisable to work only cement and sand with water into mortar, then first adding the crushed stone to the finished mortar. But usually the different materials are placed in the mixer at the same time; labor is thus saved, but less strength is obtained. Also see Cement. 1914. p. 152.

Concrete mixing machines may be classified as follows:--

a. machines working intermittently, the so-called charge mixers. ² (The material is received and discharged successively, may be mixed longer or shorter; reliable mixing, but slower.

Note 2.p.63. The machines mentioned under 1 to 4 (free falling mixers) properly differ only in the manner of discharging.

1. Free falling mixers. (Simple attendance and cheap, mostly hand labor, but with small capacity for production).

Free falling mixer of F.B.Gilbreth. Boston. Mass.

Free falling mixer of W.Daum. Miltenberg-a-M. (Pavaria).

Free falling mixer of Leipzig Cement Industry, Dr Caspary

and Co. (Inclined drum).

2. Free falling mixers with vertical drum. (Materials mixed in a rotating drum and continually falling).

Mixer of Gauhe, Gockel & Co, Oberlahnstein.

Mixer of Tobler, Borsigwalde near Berlin.

Mixer of General Architectural Machine Co. Leipzig.

3. Free falling mixers with a mixing drum rotating about a horizontal axis (mixing shovels on internal surface of drum).

Ransome mixer made by Deaustsch & Co. Berlin. (Drum holds 1200 litres, daily product up to 320 m^3).

4. Free falling mixers, where the mixing drum is tipped a about an axis perpendicular to its axis of rotation for discharging.

Smith mixer, Drals Works, Mannheim-Waldhof. (Drum holds up to 900 litres, daily product up to 360 m^3 , with 40 fillings per hour.).

5. Fixed horizontal mixing trough with discharging valve in bottom. (Trough open at top, rotary arms on horizontal shaft).

Kunz mixer of Royal Bavarian Foundry, Sontsafen in Algau. (Very productive machine, cheap and reliable in operation).

Mixer of Wolf & Co. Guben.

Mixer of Gauhe, Gockel & Co. Oberlahnstein.

6. Mixing troughs, that can be discharged by rotation about their horizontal axes. (Mostly with stirring arms on a vertical axis).

Hüsen mixer, made by Beyer & Zetsche in Flauen. (Mostly fixed for factory use).

Mill of Birich Co. Hardheim.

b. Machines working continuously. (Materials constantly & discharged at one end and received at the other; less reliable work.

1. Machines with horizontal fixed mixing trough and spiral mixing shovels, which require the materials regularly while the machine is running.

Mixer of Hüniger & Leyer, Düsseldorf-Derendorf.

2. Machines with inclined mixing trough. (Good for small work).

Mixer of Gauhe, Gockel & Co. Oberlahnstein. (Product 40 to 60 m^3 with 1.5 to 2 H.P.).

3. Machines with vertical fixed or rotating mixing cylin-

cylinder. (Also good for small work).

Mixer of Gaune, Gockel & Co. Oberlahnstein. Product as for 2; with hand power, 15 to 20 m³.

Mixer of G. Hiller, Dresden.

Funnel mixer of Leipzig Cement Industry, Dr. Gaspary & Co.

4. Machines with inclined rotary mixing drum.

Mixers of Anderson & Sons, Detroit; also Samelins, Rotterdam.

5. Machines with rotary mixing drum, where the materials are continuously moved from one end to the other by shovels, arms, etc. fixed on the internal surface of the drum.

Mixer of M. Kraus, Munich.

265 The most appropriate choice of a mixer first depends on how much concrete must be made hourly. Small mixers are dearer in proportion and are not economical in use. The machines mentioned under 2 are intermittently working, where the drum is filled with the materials, mixed, and the completed mixture afterwards discharged, are best suited for reinforced concrete work. The reason for this is, that one is able to continue the mixing process until the desired result is obtained. Continuous mixers come into consideration chiefly for great masses of foundation concrete.

Generally the preference should be given to such machines, that not only mix rapidly but at the same time thoroughly knead the concrete. The highest resistance is then obtained, which advantage should decide against the disadvantage of a somewhat greater driving power. Further points: easy and quick use, both in filling and discharging; good regulation of the water added; no falling of materials outside, which makes the parts of the machine dirty; least annoyance from chains and belts; small weight (small cost of transportation); simple cleaning; small replacement of all moving parts¹ (stirring arms, etc.). Finally, it is also preferably under some circumstances, if the machine be so constructed, that the materials while mixing remain visible to the workman.

Note 1. p. 85. Lean mixtures with large additions of crushed stone of coarser sizes naturally cause the greatest repairs and further lessen the producing capacity of the machine.

In regard to the use of machines for cast concrete, see B & E. 1914. p. 49.

As an example of a mixer is shown in Fig. 17 a machine of the

the Royal Bavarian Foundry at Sonthofen (system Kunz), that ~~it~~ has been much used for 20 years for reinforced concrete structures. This machine has a capacity of 150 litres and produces 5 to 8 m³ per hour. It belongs to a class a; a fixed open drum, a system of kneading by mixing arms with movable mixing shovels, discharging through a valve. The process of mixing is under the eyes of the attendant workman and the foreman. The compulsory thoroughness of the mixture shortens the time of mixing considerably, and increases the producing capacity of the machine. The machine is frequently driven by an 8 H.P. benzine motor, or on larger buildings by an electric motor or a portable steam engine. Frequently attached -- for buildings -- ^{is} ~~are~~ a concrete hoist. The concrete falls from the drum into a trough, and then passes without further attendance, self discharging in all stories. This arrangement has already worked to a height of 50 m at building sites.

The just described machine of the Sonthofen Foundry is that most common for reinforced concrete buildings. But the foundry also builds machines of 3.5 to 25 or even 40 m³ product per hour, indeed with and without lifts, as well as with and without attached benzine or electric motors.

B. Mixing Proportions, Materials used and Weight of Volume.

One must distinguish between "rich" and "lean" mixing, according to whether less additions are made to the cement (rich mixture) or more are added to relatively less cement (lean mixture).

For the choice of the mixture of a concrete, which is to be employed for a definite portion of a structure, the demands are to be satisfied, that relate to the strength and density to be established in the structure. But the strength of a concrete is then naturally produced by the good qualities of the raw materials, i.e., of the cement, sand, aggregate (crushed stone, gravel), as well as the water.

The good properties are both chemical as well as physical; the former come into consideration for cement and water, the latter play a part in all raw materials for concrete; these are particularly the following: the fineness and purity of the cement, the purity and temperature of the water, and finally the crushing resistance, hardness, structure, size and form of grains, and nature of the surface of the sand and of the aggre-

aggregate.

Since then these numerous chemical and physical properties combine in the most varied groupings, and thereby may influence the strength of the concrete, it is easily seen, that a numerical assumption of the mixing proportions of a concrete of a required determinate strength is not indeed possible.

The determination whether a certain concrete possesses the necessary compressible resistance for a certain part of the building thus remains entirely a matter of experiment, which is to be made in every case with great care and repeatedly. Especially (see Section III G). The strength is generally so much less, the greater the quantity of aggregate and the less the content of cement.

It is even frequently mixed too rich, particularly for great masses, like foundations, retaining walls, etc. Even very lean concrete is better and stronger against compression and tension than the best masonry in lime mortar. No fixed mixing proportions should be prescribed beforehand, but only definite strength is required with a definite factor of safety.¹ By conscientious selection of the materials equal strength may be obtained by leaner mixtures, than by a capricious choice with considerable richer mixtures.

Note 1. p. 68. For example, a prescribed mixture of 1 cement + 2 sand + 5 crushed stone is only to be regarded as usable, when in the 5 parts of crushed stone is found at least 35 p.c. as stone chips 7 to 25 mm in size.

The mixing proportions usual in practice, and most conveniently numerical, are not always those most favorable to economy, and naturally can make no claim to perfect density of the concrete.

The future density of the concrete,² is assured, if all cavities existing in the aggregate are filled by mortar (cementing material), and thus the cementing material itself is dense, i.e. all crevices in the mortar sand are filled by cement. By the representation in Fig. 18, it is evident, how such a definite quantity of concrete is composed of cement, sand and crushed stone; cement and sand particularly serve to fill the crevices of the aggregate.³ The density of a concrete or of a mortar can generally be expressed by the formula:—

$$d = \frac{\text{cementing mass}}{\text{crevices}}$$

Note 2. p. 68. The density plays a great part in certain buildings, for example in water reservoirs, tunnels under streets, etc., but particularly for reinforced concrete structures, and indeed in reference to protection of the reinforcement from rust.

In most sands in use the limit of density of the mortar is given by about the mixture 1:3.

Note 3. page. 88. Attention should be called here to the fact, that indeed the most common procedure of determining the cavities by filling with water is not entirely free from objections; as a rule it gives too small values on account of air bubbles, that adhere to the sand examined. It is safer to obtain the x volume of crevices from the volume weight of the sand (shaken down strongly) and the specific gravity of the same (averaging 2.65).

In a loose mass are contained crevices:--

In cement about 52 p.c.

In sand about 30 to 35 p.c.

In gravel about 35 to 40 p.c.

In crushed stone about 40 to 50 p.c. (A smaller addition of crushed stone, then/less product and dearer concrete).

If all crevices are exactly filled with the mortar, numerator and denominator of the expression for d are equal, and one speaks of a mortar or concrete with the density $d = 1$. Hence a mass is not dense, if the amount of crevices exceeds the amount of mortar existing.

Practically, in order to be safe and to avoid the direct contact of pieces of aggregate, care is taken to give the mortar a certain excess beyond the quantity required for $d = 1$, and this excess amounts to about 15 p.c., i.e., one uses 1.15 times the quantity of mortar, that would produce dense concrete, thus obtaining a concrete with the density $d = 1.15$, where all pieces of aggregate are coated by a layer of mortar.

It is usual to indicate concrete mixtures by the form:-- 1: m:n, for example 1:3:5, this denoting that the mixture consists of 1 cement + 3 sand + 5 aggregate (crushed stone, etc.). With the use of gravel sand, i.e., gravel containing sand in the proper proportions, one writes 1:m, for example, 1:7.¹ The parts of the different materials are given in volumes (litres). The proposal to mix by weight at the building site (as usual and proper in laboratories), can only be agreed to conditionally,

as well as the proposition to measure the cement by weight and the additions by volume. The volume weight of the aggregate varies with its mineral composition, and according to its actual content of water, and the volume weight of the cement according to the degree of snaking together. Different volume weights of the cement (see p. 68) would lead to considerable irregularities. The following weights may serve for calculations.

Sand 1500 kil/m³ of a loose mass.

Coarse gravel 1550/m³ of a loose mass.

Fine gravel 1570 kil/m³ of a loose mass.

Coarse crushed stone 1300 kil/m³ of a loose mass.

Fine crushed stone 1320 kil/m³ of a loose mass.

Portland cement poured 100 to 120 (averaging 120) kil/400 litres.

Portland cement snaken 160 to 210 (aver. 190) kil/400 litres.

Portland cement filled 110 to 130 (aver. 125) kil/400 litres.

Note 1. p. 68. In a given case for gravel sand, it must be determined by sifting test what proportion exists between sand and gravel. (Also see p. 44).

Note 1. p. 70. So far as the measurement of cement by volume occurs, it is assumed that without falling, the cement is poured into the measure (not shaken down). To compute the volume from the average weight of Portland cement, this may be taken at 1400 kil/m³.

The statement of the cement added in parts by weight has however been increasingly adopted in recent times, especially in France, South Germany and Switzerland. Thus the Swiss Regulations say:-- (Art. 13):-- The concrete is to be mixed in parts by weight for Portland cement, and in parts by volume for gravel and sand; for making standard concrete for 1 m³ of mixed gravel and sand, i.e., for 0.8 m³ gravel and 0.2 m³ sand, is to be used 300 kil Portland cement.

Materials used for 2 sacks. Materials for 1 m³ concrete.

Mix.	Cem.	Sand.	Grav.	Ston.	Prod.	Sem.	Sand.	Grav.	Stone.
1:2:3	2, 50	.145	---	.215	.290	350 k	.500	---	.750
1:2:4	..	.145	.290	---	.350	285	.400	.625	---
1:3:4.5	..	.215	---	.325	.430	234	.500	---	.750
1:3:6	..	.215	.430	---	.495	202	.435	.870	---
1:2:3	2, 56.7	.165	---	.245	.330	Same as above.			
1:2:4	..	.165	.330	---	.400				
1:3:4.5	..	.245	.495	.370	.495				
1:3:6	..	.245	.495	---	.565				

Note 2. p. 70. "Product" is the ratio of volume of the finished concrete mass to the sum of the volumes of the components. Generally the product may be assumed at 70 to 75 p.c. of the mass of the components.

p. 71} If a concrete with definite mixing proportions can be recognized by thorough experiments as corresponding to the requirements of an existing special structural case, then is required for estimating and obtaining the raw materials a knowledge of the required different materials for a unit volume of completed bearing concrete (1 m³), cement, sand, aggregate (gravel, crushed stone, etc.). It is generally advisable to take the volume of gravel or crushed stone about twice as great as that of sand, assuming that this refers to grains of different sizes. But if the crushed stone (especially that made by hand) has quite uniform sizes of grains, then the volume of crushed stone is taken only 1 1/2 times the volume of sand. In the Table on p. 70 are given for both cases the volumes of materials for a mixture with 2 sacks of cement. ¹

Note 1. p. 71. Also see Table on p. 73, the line 20 p.c. is to be used here.

For example, if for a building a total of 250 m³ of concrete is to be mixed 1:3:6, then come in question the volumes of the different materials.

Cement = 250 × .202 kil = 50,500 kil.

Sand = 250 × 0.435 m³ = 109 m³.

Gravel or mixed crushed stone = 250 × 0.670 = 218 m³

In the following Table is given for use of a gravel sand mixture. Cement + sand and gravel (1:2) + water materials for Mix. . produces in tamped concrete .1 m³ concrete:

			Cem. kil	Sand	gravel. m ³
1:1	50 k + 36 l + 12 l =		54 l 925		0.667
1:2	50 k + 72 l + 13 l =		83 l 625		0.868
1:3	50 k + 108 l + 15 l =		112 l 450	2	0.964
1:4	50 l + 144 l + 18.6 l =		141 l 355		1.021
1:5	50 l + 180 l + 22.2 l =		170 l 295		1.059
2:6	50 l + 216 l + 25.8 l =		200 l 250		1.080
1:7	50 l + 252 l + 29.4 l =		230 l 220		1.091
1:8	50 l + 288 l + 33.0 l =		263 l 190		1.095
1:9	50 l + 324 l + 36.6 l =		296 l 169		1.094
1:10	50 l + 360 l + 40.2 l =		330 l 151		1.091

$$1 : 11 \quad 50 \text{ k} + 396 \text{ l} + 43.8 \text{ l} =$$

$$365 \text{ l} \quad 137 \text{ l.055}$$

$$1 : 12 \quad 50 \text{ k} + 432 \text{ l} + 47.4 \text{ l} =$$

$$400 \text{ l} \quad 125 \text{ l.060}$$

P. 72 Note 2. p. 71. For mixing 1:3, better take 500 kil cement as an average value.

P. 72 For reinforced concrete (floors, piers, etc.) as a rule is taken as mixing proportions 1:4 to 1:6, according to the statistical requirements of the member. Frequently at least 300 kil per m^3 is officially prescribed. (See p. 70, Note 1). The cost of the concrete in reinforced concrete construction mostly plays no such important part, that the economy of the structure will be strongly affected by a rather too great addition of cement. It is first of all how the concrete is later stressed. For a maximum stress of only 30 kil/cm² already suffices a mixture of 1:6 to 1:7. But the mixture may be leaner in no case, with regard to the safety of the reinforcement from rust. For tamped concrete foundations are naturally employed leaner mixtures. (See Fig. 19).

A method of calculation of the materials required and adapted to the proportions occurring in practice may proceed on the basis of the following considerations:— According to his experience the specialist knows by the use of the different aggregates approximately, with what p. c. loss in tamping he must reckon. One can generally count $\frac{1}{4} \text{ m}^3$ of finished concrete as requiring 1.3 to 1.5 m^3 of materials. By $\frac{1}{4}$ loss is to be understood the sum of the amounts by which the loose mass shrinks in mixing by filling the existing crevices and by the tamping. It is here to be assumed, that the ^{cement} ~~mass~~ entirely disappears in the voids, thus having no effect on the later total volume. For example, if one mixes 5 m^3 with 1 m^3 cement, then the mixed and tamped mass does not amount to 6 m^3 , but only to about 4.5 to 5.0 m^3 . Again designate the mixing proportions by 2:m:n and the p.c. of shrinkage by ϕ , then for 1 m^3 of tamped concrete are required the quantity of materials as follows. ¹

Note 1. p. 72. See Kutsche, "Material required and Density of Concrete Mixture, assuming that the aggregate can be tamped together."

$$M_2 = \frac{1}{(m+n)(1-\phi)} = 3\phi : M_3 = m\phi : M_1 = n\phi.$$

P. 73 For the usual concrete mixtures the shrinkage amounts to 15 to 25 p.c.; thus ϕ lies between 0.15 and 0.25. For example, for 1 m^3 concrete, 1:2.5:5, with $\phi = 0.15$ would be required.

$$M_2 = \frac{1}{(2.5 + 5)(1 - 0.15)} = \frac{1}{8.88} = 0.157 \text{ m}^3 \text{ cement.}$$

$$M_3 = 2.5 \times 0.157 = 0.393 \text{ m}^3 \text{ sand.}$$

$$M_4 = 5 \times 0.157 = 0.785 \text{ m}^3 \text{ aggregate.}$$

Gravel sand shrinks less in tamping than crushed stone or g gravel, when separate before mixing. For gravel sand mixtures, one may count on a shrinkage of 13 to 25 p.c.: for separately added gravel or crushed stone, 25 to 30 p.c.

Mixture.	$\phi = 0.80$			$\phi = 0.25$			$\phi = 0.20$			$\phi = 0.15$		
1:2:n.	Cem.	Sand	Agg.	Cem.	Sand	Agg.	Cem.	Sand	Agg.	Cem.	Sand	Agg.
1:1:1	714	714	714	667	667	667	625	625	625	588	588	588
1.5	571	571	557	533	533	500	500	500	500	471	471	707
2	476	476	952	444	444	888	417	417	834	392	392	784
2.5	408	408	1020	381	381	953	357	357	893	336	336	840
3.	357	357	1071	333	333	999	313	313	939	294	294	882
3.5	~~317~~	317	1110	296	296	1036	278	278	973	261	261	914
2:1.5:1.5	476	714	714	444	666	666	417	626	626	392	~~598~~	588
2	408	612	816	381	572	762	357	536	714	336	354	672
2.5	357	536	893	333	500	833	313	470	783	294	441	735
3	317	476	951	296	444	888	278	417	834	261	392	783
3.5	286	429	1001	267	401	935	250	375	875	235	353	823
4	260	390	1040	242	363	968	227	341	908	214	321	856
1:2:2	357	714	714	333	666	666	313	626	626	294	588	588
2.5	317	634	793	296	592	740	278	550	695	261	522	653
3	286	572	858	267	534	801	250	500	750	235	470	705
3.5	260	520	910	242	484	847	227	454	795	214	428	749
4	238	476	952	222	444	888	208	416	832	196	392	784
4.5	220	440	990	205	410	923	192	384	864	181	362	815
1:2.5:2.5	286	715	715	267	668	668	250	625	625	235	588	588
3	260	650	780	242	605	726	227	568	681	214	535	642
3.5	238	595	833	222	555	777	208	520	728	196	490	686
4	220	550	880	205	513	820	192	480	768	181	453	724
4.5	204	510	918	191	478	860	179	448	806	168	420	756
5	190	475	950	178	445	890	167	418	835	157	393	785
1:3:3	238	714	714	222	666	666	208	624	624	196	588	588
3.5	220	660	770	205	615	718	192	576	672	181	543	634
4	204	612	816	191	573	764	179	537	716	168	504	672
4.5	190	570	855	178	534	801	167	501	752	157	471	707
5	179	537	895	167	501	835	156	468	780	147	441	735
5.5	168	504	924	157	471	864	147	441	809	138	414	759

1:3:6	159	477	954	148	444	888	139	417	834	131	393	786
1:3.5:3.5	204	204	714	191	669	669	179	627	627	168	588	588
4	190	665	760	178	623	712	167	585	668	157	550	628
4.5	179	627	806	167	585	752	156	546	702	147	515	662
5	168	588	840	157	550	785	147	515	735	138	483	690
5.5	159	557	875	148	518	814	139	487	705	131	459	721
6	150	525	875	148	518	814	139	487	705	131	459	721
7	136	486	952	127	445	889	119	417	833	112	392	784
1:4:4.5	168	672	756	157	628	707	147	588	662	138	552	621
4	179	716	716	167	668	668	156	624	624	147	588	588
5	159	636	795	148	592	740	139	556	695	131	524	655
5.5	150	600	825	140	500	770	132	528	726	124	496	682
6	143	572	858	133	532	798	125	500	750	118	472	708
6.5	136	544	884	127	508	826	119	476	774	112	448	728
7	130	520	910	121	484	847	114	456	798	107	428	749
8	119	476	952	111	444	888	104	416	832	98	392	784
1:5:5	143	715	715	133	665	665	125	625	625	118	590	590
6.5	130	650	780	121	605	726	114	570	684	107	555	642
7	119	595	833	111	555	777	104	520	728	98	490	686
7.5	114	570	855	107	535	803	100	500	750	94	470	805
8.	110	550	880	103	515	824	90	480	768	90	450	720
9	102	510	918	95	475	855	89	445	801	84	420	756
10	95	475	950	89	445	890	83	415	830	88	390	780
1:6:6	119	714	714	111	666	666	104	624	624	98	588	588
7	110	660	770	103	618	721	96	576	672	90	540	630
8	102	612	816	95	570	760	89	534	712	84	504	673
9	95	570	855	89	534	890	83	498	780	78	468	780
10	89	534	890	83	498	830	778	468	780	74	444	740
11	84	504	924	78	468	858	74	444	814	69	414	759
12	79	474	948	74	444	888	70	420	840	65	390	780
1:7:7	102	714	714	95	665	665	89	623	623	84	588	588
8	95	655	760	89	623	712	83	581	664	78	546	624
9	89	623	801	83	581	747	78	546	702	74	518	666
10	84	588	840	78	546	780	74	518	740	69	483	690
11	79	553	869	74	518	814	70	490	770	65	455	715
12	75	525	900	70	490	840	66	462	792	62	434	744
1:8:8	89	712	712	83	664	664	78	624	624	74	592	592
9	84	672	756	78	624	702	74	592	666	69	552	621
10	79	632	790	74	592	790	70	560	700	65	520	650
11	75	600	825	70	560	770	66	528	726	62	496	682

p. 74. The Table just given (p. 73, 74) contains the quantities required for a great series of mixing proportions for the values $w = 0.30, 0.25, 0.20, 0.15$. If the cement is to be expressed in kil, then the tabular values are to be multiplied by 1.4. (See p., 25, 58, 70).

Further see Hoerner, Table of Cost of Cement Mixtures. Handbook for Reinforced Concrete Construction. 2nd edition, Vol. 2. Edding & Schumann. Portland Cement and its Use in Building. Castner, Cement and its rational Use for Building Purposes. Dieck, Mortar, Materials required, and Tables of Prices. Safir, Nature, proper Mixing proportions and Products of Hydraulic Building Materials. Unna, Determination of rational Mortar Mixtures. Further, see B & E. 1910. p. 396; 1912, p. 140; 1914, p. 92, 128, 178.

The volume weight of concrete depends on the selection of materials, on tamping and on the water added, less than on the mixing proportions. ¹ It is in general higher, the greater the tamping and the richer the mixture. Test cubes as a rule show a greater volume weight on account of the stronger tamping, than would correspond to the concrete of the structural member concerned.

Note 2. p. 75. For pumice and cinder concrete too small values are mostly given; one has even established volume weights of 1500 to 1700 kil/m^3 for these kinds of concrete. The Prussian Regulations on loads to be employed for buildings (1910, Jan. 31) give only 1000 kil/m^3 as average values (see Appendix), which can always be taken for filling purposes.

Note 1. p. 75 For ordinary ground concrete about the following values are to be taken.

Mixture	1:3	1:5	2:7	1:10	1:12	1:15
Vol. Wt. in kil/m^3	2300	2250	2200	2170	2150	2140

One may usually compute with the following values:--

Cement + gravel (gravel sand)	2300 to 2300 kil/m^3	2200 Aver. kil/m^3
Cement + crush. granite	2000 to 2500	2300 ..
Cement + basalt crushed	2200 to 2500	2400 ..
Cement + lime or sandstone	2000 to 2200	2100 ..
Cement + crush. bricks	1500 to 2000	1800 ..
Cement + pumice gravel	1000 to 1500	1300 ..
Cement + coal cinders	800 to 1500	1200 ..

On volume weights of reinforced concrete, see Section VI. 1

P. 76. C. Wooden Forms and Centering.

For the purpose of concrete work is first to be arranged a wooden centering, that causes considerable expense to the contractor executing the work. For wood is in itself a comparatively dear building material; further by cutting and using, it is injured more than is the case for working scaffolds and centerings for masonry structures. ¹ Furthermore a great portion is always lost during the building, since stealing of some lumber is scarcely to be prevented. For simple structural forms the cost of forms amounts to 10 to 20 p.c., but for large buildings often to 50 p.c. and more of the entire cost of the building. By proper and skilful^{ly} using again the lumber on the building site, the work of forms may be cheapened. A circular saw driven by power can be of great advantage in the construction of the forms for reinforced concrete construction, especially if this circular saw is driven by the same motor as the concrete mixer or the existing structural hoist.

Note 1. 76. After being used several times, the lumber only has the value of fuel. With round poles one can count on using 10 times in favorable conditions, as sheathing boards for plane floors 4 times, or 5 times for walls.

An important American Contracting Co. in reinforced concrete construction proceeds as follows, in order to spend as little money as possible for its forms for tamping:-- it has prepared from the centerings of those buildings, that it has executed with the best results, models at 1/8 natural size, which serve for instructing its officers. When the Co. undertakes a new building, it also has a model constructed for the centering for tamping, for which are utilized all previous experiences, and its officers and workmen are required to carefully examine this model and to make suggestions for simplifying or cheapening it. These suggestions are thoroughly tested for their usability and are rewarded by prizes in favorable cases. The Co. thus best utilizes the knowledge its officials have acquired in other construction companies.

To save cost of centering an estimate is also made for the entire building -- or at least for two stories -- taking the same sizes of piers, dimensions of beams and vaults. In this way the completed wooden centering can be used several times. In the lowest story the dimensions of the supports and beams

must naturally be as small as possible. The greater amount of steel thereby used in the lower, and of concrete in the upper stories is again compensated by the possibility of using the same centering several times. ¹-- One should at least consider the avoiding of all small differences in the girders and supports of a floor, already in the design.

Note 1.p.77. It always appears questionable, whether in rapid working, the timbers will always be used again in the correct places in the upper story. If one desires to produce actual savings, a particularly careful oversight is necessary.

Also attention is to be called here to this, that with the existence of so much centering and form lumber a fire may easily break out, which may have the most important effect on the insufficiently hardened concrete. The Vienna city building office therefore only permits electric lights for artificial lighting, and requires that benzine motors and smiths' fires be placed outside the building, all wood rubbish to be collected during the process of building, and removed daily from the structure. Also see "Burning of a reinforced concrete structure on centering." Ann. B. 1911. p. 265.

For extensive works, where similar structural parts frequently occur (for example, floors, ribs), one can previously make in the working yard great panels, form boxes, etc., that can be used over again, and thus substantially reduce the work of preparation. Naturally must the centering always be so connected together, that easy removal of the side parts is possible. For frequently repeated structural parts are often employed iron forms, ¹ especially for such parts as are made in the factory. (Visintini, Siegwart systems, etc.). Bolt holes and the like are to be formed by gypsum or wooden pins, that can be drawn out a few hours after tamping.

Note 1.p.78. Advantages of iron forms:-- use for many times, fireproof, more rapid removal of forms, production of smooth concrete surfaces. Further see B & B. 1912. p.38.

The forms must have sufficient strength and supporting capacity to be able to support the shocks of tamping as well as the weight of the concrete and of the laborers without the appearance of any change in shape. ² One counts for forms, according to whether the architectural members are light or heavy, 3 to 1 1/2 times the weight of the concrete. In a given case,

it is recommended to arrange a slight convexity of the floor centering to equalize the deflections occurring under the weight of the concrete. The fixing of the forms must present as little difficulty in handling as the removal; An interchange of the centering planks must proceed easily and conveniently. The use of nails is to be limited as much as possible; screws and clamps are preferable. For large structures and necessary a preliminary drawing and calculation of the centering framework. If one desires to employ unused lumber, it is often advisable to send the drawings to an experienced carpenter for estimate and for later supplying the framing put together in a mill; ³ (better use of pieces and waste, cheaper working in the mill). The salvage plays a great part in the forms. Already in the design of a structure must attention be devoted, as far as possible, to avoiding shapes difficult to make in forms, and therefore rather to select shapes that are repeated.

Note 2.p.78. Since in reinforced concrete construction most accidents result from imperfect forms, the rules in reference thereto have been substantially extended from the earlier ones.

Note 3.p.78. But the work can easily be made unnecessarily expensive; for the master carpenter has learned his trade by the erection of permanent wooden structures. It is more economical to have the plans for fixing and removing the forms prepared by the supervising engineer.

The sheathing planks usually have parallel edges as left by the saw, are sufficiently thick (3 to 6 cm),¹ and are to be firmly joined, so that by tamping no splits and above all no deflections occur. Mostly pine lumber is used, indeed in lengths of 4 to 6 m; Shorter planks are obtained by cutting. Planks previously used for forms must naturally be cleaned off. Before the placing of the concrete, then the completed forms are to be cleared from the wooden pieces, sawdust, dirt and the like, are to be well wetted, so that the dry wood shall not absorb from the concrete the water necessary for setting.

Note 1.p.79. Thin boards must be more frequently supported, which requires more lumber for supports and more wages. For simple plane floors already suffice planks 33 mm thick; for architectural forms with much cutting, it is best to take boards about 26 mm thick, for girders planks about 5 cm thick. (5 x 15 cm). The thicker the lumber, the firmer and more resis-

resistant the centering, but so much the greater is the loss by cutting.

The centering is usually close jointed, since the existence of too open joints is at once noticeable on the smooth surface of the concrete. But on the other hand is to be taken into account the springing and ~~curving~~^{warping} of the planks in consequence of the effect of the wet concrete mass. Therefore it is advisable, especially for great surfaces, to arrange for open joints up to 1 cm wide and to set the centering properly beforehand. The projection of the concrete through the joints is best prevented by the use of cloth, which at the same time absorbs the superfluous water. Matched wood would be too dear, and would soon be broken off. ² It is quite difficult to produce an entirely smooth visible surface, since all irregularity in the ~~forms~~^{forms}, such as bends, knots, etc., exert a disturbing influence.

Note 2.p.79. Matched lumber comes into consideration for very wet concrete. (Cast concrete). (See p. 42).

For supporting the centering for the slabs and beams on the floors or ground beneath serve struts (timbers, posts), ~~between~~^{but} which are necessarily to be so arranged, that the centering, after removing these supports, can be taken away without trouble. For frequently must the sides of the forms be removed soon after the setting of the wet concrete, for their use again at other places, while the timbers are necessary for a time to support the entire centering. ¹

Note 1.p.80. See Section IV, 5.

The struts as in Fig. 35 are to be set on oaken wedges of proper shape, and these are to be carefully watched during the concreting work. When set on the ground (cellar story) a sufficiently large block is to be placed under the wedge to distribute the pressure. ² Generally, one counts about 1 strut to each m^2 of floor area, for a strong plank centering, a strut to 2 m^2 , for wooden or iron box forms a strut to 3 m^2 or more of floor area. Usually round logs of 8 to 15 cm diameter are taken. (Mostly 10 or 12 cm diameter). Squared timbers (8 x 8 cm to 14 x 14 cm) are seldom employed on account of the higher cost. Long struts must have greater diameters on account of the danger of buckling; otherwise bracing both ways, which is always proper, is here certainly necessary.

Note 2.p.80. The area of the bearing surface must correspond

to the ~~actual~~ ^{actual} nature of the ground. The earth may be wetted by the water used in concreting, which may result in the sinking of the framework.

For the supports must be employed whole timbers if possible; if a single splice be unavoidable, then must the struts be securely and firmly connected together at the joints. ³ Simple bolted joints are not always to be recommended, and steel tubes slipped over each other ¹ are only used for very frequent use. There are various patented arrangements for adjustable struts, that make possible rapid and simple handling. The Fix strut ~~holder~~ ^{holder} is illustrated in form and arrangement in Fig. 20. Any slip of the timber is thereby prevented, since on account of the clamp, it cuts into the lower end of the strut.

Note S.p.20. The Kurlenberg Regulations prescribe the following:-- "For the centering for ^{clear} floor construction supported by struts, spliced supporting struts may be used for only 1/3 of the whole, and only be set alternately with through struts. Struts spliced several times and those under 8 cm thick at top and are not allowed. For a length of 5 to 8 m the struts must be braced both ways."

Note 1.p.81. On an extensive framework of steel pipes see P & S. 1912. v. 346.

The supports for greater heights (vaults, domes) must be by frameworks in wood or steel. But for floors one may proceed as in Fig. 21, or use supporting trusses as in Fig. 21, which extend from wall to wall. In such manner are saved the dear struts, a usable room is retained below, and one does not have to reckon on unequal yielding of masonry and struts afterwards.

To remove larger vault or framed forms serve sand tins, screws and the like. If the framework is exposed to the influence of storms, then naturally particular emphasis must be placed on a proper opposition.

Chamfering the angles as in Fig. 39 b, p. 91, prevents the breaking of the angles in removing forms from beams and piers. Small triangular wood strips 3 to 5 cm in width on sides are used, that can be rapidly cut out with the circular saw.

If after the hardening of the concrete, it receives no plastering, then the sides of the planks coming in contact with it are to be planed smooth. Then only lumber free from knots is to be used, or places with knots are to be smeared with gypsum.

To make possible close jointing of the planks, it suffices to plane their edges. Besides to prevent the concrete from sticking, and an imprint of the grain of the wood in all cases, it is frequently recommended to coat the planed sides of the planks either with mineral oil, soap, whiting (in thin paste) or milk of lime (also shellac), or to place an intermediate layer of paper, felt or cloth. By coating with soap, the pores of the wood are filled. A coating of thick oil prevents wetting the wood, ^{more durable} and in this way makes it ~~possible~~. Metal sheets, even zincked iron, are not properly advisable, since it is impossible to avoid bulges in the sheets. Yet no sufficient security is required for a perfectly smooth and plane surface of the concrete; the means mentioned also make more difficult and costly the work of concreting in a relatively high degree. An application of plaster afterwards may be preferable in many cases, at least in the domain of buildings. ² Naturally planing of the planks is unnecessary then, since the rough and uneven surface of the concrete produced is best suited for the firm adhesion of the plastering. ³

Note 1.c.82. Good results are also obtained with oiled paper tacked on. (Extra cost about 0.2 mark/m² of surface. Just as good service results from painting the surface of the planks with a mixture of 1 linseed oil + 3 petroleum.

Note 2.p.82. See Section III, P.

Note 3.p.82. Yet in this respect bad experiences have already occurred, especially for structural members exposed to considerable vibrations and shocks:-- the plastering mortar partially flakes off, whereby the entire structure looks worse, than if it had remained without plastering.

Two methods should be mentioned, that may partly make possible an avoidance of the quite ^{expensive} ~~expensive~~ forms for reinforced concrete structures. First some inventions have the aim of tam-
ping on the level ground, already before their proper use, the supporting structural members, particularly columns and beams, in order to set them in their proper places after the expiration of some weeks. They will then give a strong and suitable support to the necessary centerings for walls and floors. Such a method of working reduces in a high degree the cost of a reinforced concrete structure, and allows it to be continued at any season of the year, but it excludes the previously intimate

connection of the different members completed at different times. A previous construction of these appears to be only possible, if in statical respects they have nothing in common with the adjacent walls and floors. The disadvantages of this method of working can scarcely outweigh the advantages of it — the possibility of a very rapid construction of the building, and the easy testing of the completed members in the workshop.

The second method for simplifying the centering is to give the framework of the centering the form of a self-supporting structure. The boards and planks are suspended from the steel skeleton, that must be made correspondingly strong, and thereby the number of supports otherwise required are essentially reduced, sometimes to none. ¹ Advantageous is then such a procedure, if no sufficient space exists for a construction of a framework for the centering. Yet it may be so far objectionable, since the solid enclosure of the steel by the wet concrete then occurs, when it suffers notable deflections by the load of the suspended centering. If the steel members are to be so strongly dimensioned, that such deflections are excluded, then would the danger result, of making the entire structure too heavy, rather increasing than reducing the cost of the building, corresponding to the greater mass of materials.

Note 1. p. 88. Also see the Melon system, Kersten. *Armed Bridges*. 3rd edition, p. 188, 224. Pohlman system, Leschinsky. (Kersten. *Reinforced Concrete Construction*. Part II. 7th edition, p. 43.)

In regard to the strength of the centering timbers for large structures, reference is made to the experiments at the erection of the New Main Railway Station at Leipzig. (Contribs. 1912, p. 76). Oak under compression parallel to fibres showed a resistance of 306 kil/cm², but Australian hardwood (Moa wood) on the other hand, 543 kil/cm². While oak under compression perpendicular to fibres showed only 67 kil/cm², for Moa wood was found a resistance of 217 kil/cm². On the ground of such results was employed for struts German oak, but for different wooden lintels and sills lying at the tops and bases of these struts was used Australian Moa wood.

Further see the loading experiments with wooden frameworks made for the falsework of the Sitter Bridge (near S. Gall) on Lake Constance-Toggenburg Railway; "Steel Construction" 1910.

p. 419. "Journal for Deep Construction". 1912. p. 37.

In the following will be given some examples of centerings for floors, piers and walls.

1. Centerings for Slabs.

For the construction of floors between I-beams, it first depends on whether the floor rests on the top or bottom flanges — so far as concerns the centering. For the first case mentioned it is simplest to place the supporting beams of the centering planks on the lower flanges, as shown in Fig. 22. The planks are 3 to 4 cm thick. It is advisable to place wedges between the lower flange and the timbers.

For floors lying below, the supporting beams may be fastened by twisted centering clamps, as in Fig. 23.

Safer is the suspension of the centering by special hangers, which likewise rest on the bottom flange, as Fig. 24 shows.

Flat bars set on edge receive the load and are secured by wedges from displacement. If the flat bars are sufficiently long, they can be used for different floor spans.

In Fig. 25 is shown a hanger, that can be screwed up from below. For small spans the centering lies directly on the supporting beams; for larger spans of the centering separate transverse beams are necessary. The removal of the centering follows the loosening of the nuts. The hangers are to be well greased or soaped, so that they can be readily drawn out. A similar arrangement to be screwed up from above is shown by Fig. 25. (Preferable).

Fig 27 illustrates the use of U-shaped bent hangers. This is a flat steel stirrup, whose sides extend downward, at their lower ends are holes for inserting a round pipe. The whole rests on the top pin a. The longitudinal beams directly support the centering and are held by the pin b. A wooden wedge between the top flange and pin a serves to correctly locate the arrangement of the centering. The two lower ends of the flat bars can also serve to support a hanging scaffold for the work of plastering.

Instead of the hangers may also be used heavy wire as in Fig. 28, passed around a flat bar, cut off at the underside of the ceiling after removal of the centering.

The use of previously described flat bar hangers (Fig. 24) also follows for vaulted ceilings in a suitable way, as visib-

visible in Fig. 29. The flat bars are bent according to the curve.

Fig. 30 represents an arrangement of centering for vaulted ceilings, such as introduced in the trade by Peschke Co, Zweibrücken. These are all removable steel arch forms, consisting of two separable angles. On their lower flanges they have a secure and not obstructing slide, but are entirely smooth on top. The hangers at right and left are easily adjustable to the thickness of the floor and then wedged fast. For greater spans these arches are less suitable, since they are inconvenient and dear. In all cases is recommended the arrangement of a joint at the middle of the span. But for small spans they are quite practical; they save the otherwise common wooden frames, make possible a rapid removal of the centering, and may be used again for years.

A centering for vaulted ceilings by the aid of planks (a) set on edge, which rest on the bottom flange of the beam, is shown by Fig. 31. The cavities produced at the resting places of the beams are later filled.

Recently hollow block floors have come into extensive use, as evident from Fig. 32, permitting substantial simplification of the centering work and a saving of lumber. See Part II. 7th edition. p. 22.

The slabs of ribbed floors can be formed according to Fig. 33 by planks (5 x 15 cm) set on edge at distances of 60 to 80 cm, which end in arched forms. For spans greater than 2 m, as shown in the illustration, special supports at the middle are necessary.

2. Forms for Beams and T-Beams.

Figs. 33 a, b, first show examples of the forms for supporting beams made in factories. (Visintine beams and the like). Several boxes are arranged beside each other, making it possible to make a number of beams at the same time in the workshop.

An example of a simple form of a beam with inclined ends is represented in Fig. 34. The sides (planks about 5 x 15 cm) are held together by short pieces a, stiffened at top by braces b. As soon as the ceiling panels are formed, these pieces b are naturally removed. The longitudinal planks c (5 x 15 cm) along the side walls serve to bear the beams for the slab centering. (Fig. 35 b). The sides themselves are drawn down at the ends

on account of the arches, and the short bottom planks of the arch are nailed on strips a (4×6 cm). Such forms as are shown in Fig. 34 are best framed in the work yard, then put together at the place of use. The galls for the side beams (Fig. 35 b), as a rule are only arranged when the forms of the main beams are built in place and supported.

Fig. 35 a shows the connection of slabs and beam forms as well as the required supports. The number of struts for the beam is generally arranged according to the depth of the beam; wide and deep beams (girders) require doubled struts; for the smaller beams (longitudinal beams) suffice singly arranged struts. The struts stand at distances of about 1.5 m. (P. 80).

Fig. 35 b exhibits the connection of a side beam form with a girder form. These are first placed. On both cross pieces a, a , and at the intersection with the girder form (Fig. 34) is nailed a short plank (about 5×10 cm), which there has to serve as support for the form of the side beam. The connection with the form for a pier may then follow in a similar way.

Of great practical use for the centering for flat as well as arched slab and T-beam floors between masonry walls may be the "Universal Centering" of the General Concrete and Steel Co. Berlin. (Fig. 36). This receives its entire fastening by pressure or insertion in the mortar joints. In a given case also suffices simply placing on the last course of bricks. At the places of contact with the wall and with the frame timbers, the iron is ribbed for a better hold. The height of the centering is regulated by a corresponding choice of the bearing joists and by inserted wedges. The universal framework requires no stiffening by struts. The excess in cost of the centering by more cutting of the wood is entirely avoided even for such combinations for larger spans, since the height of the framework can be chosen to correspond to the existing lengths of timbers. The advantage of this framework thus consists in a substantial saving in lumber needed and in wages. By Figs. 36 and 37 is evident the mode of use for small and large spans.

Forms for Supports.

Forms for supports are always to be so arranged, that the placing and tamping of the concrete mass may be done at an open side to be closed with the progress of the work and can be accurately observed there. For simple cases suffices a form f

for the support as in Fig. 36. Two board sides are nailed to 4 vertical corner timbers (8×8) or 8×10 cm). The wall consists of transverse pieces of boards, also nailed to the corner timbers. Walls d and c are closed, according to the progress of the work. — Another (advisable) form is shown by Fig. 39 a. At 3 sides are set vertical planks, (seen in plan as end wood), which extend to the girder and are connected together at certain distances by nailed cross pieces, as in Fig. 34. The box made thus may be constructed on the work yard, and then set up at the place of use. To hold together the frames may be used long bolts at about 1 m distances (Fig. 39 a), or doubled wires, twisted by a bit of wood (Fig. 39 b); likewise iron clamps may be used as in Figs. 39 c or 40). ¹ The fourth side of the form remains open and is occasionally closed by transverse boards, corresponding to the work of concreting. But one avoids nailing the cross boards to the longitudinal members (as in Fig. 39 d), since by nailing on the boards the freshly tamped concrete is easily again disturbed. If the form be constructed closed on four sides as in Fig. 39 e, then by the arrangement of an opening at the bottom, care is taken that all sawdust and bits of wood falling into the form during its making, can be removed before placing the concrete, as well as again checking the reinforcement in its arrangement at the bottom. The whole is held together by wooden frames; the frames are set on edge perpendicular to the form boards, and are connected together at the corners by bolts. If the angles of the piers are to be considered, then triangular strips are arranged as in Fig. 39 b.

Note 1.p.91. The connecting pieces can be alternately arranged as if rotated 90° , so that the surfaces shall be held together in two directions.

According to Fig. 39 e, h, four strong wooden supports are placed at the corners, that at distances of about 1 m are connected by strips nailed on against bending by tamping. The form construction here proceeds by pieces of boards fastened on, so that tamping can be uniformly done from two or all four sides at the same time. But this method for forms is relatively dear.

Note 1.p.92. In regard to this, see the Thorban system of forms. Berlin. Contribs. 1914. p. 4.

In all cases should one take all possible care, that in rem-

removing the form no nails are to be drawn out, and that this removal may be done easily and without snocks. (Inserted boards, bolts, clamps and the like).

If it be desired to secure the forms from bending sidewise of the filled ends, a series of strips can be arranged as in Fig. 41, which can be removed on the second day after tamping and used elsewhere.

P. 93 4. Forms for Partitions and Walls.

The usual small thickness of partitions requires the use of forms with side supports. Fig. 42 a gives an example of this. Wooden struts, set according to the progress of the concrete work between the form walls -- provide for accurately preserving the thickness of the wall. In placing the concrete only two or three boards are placed above each other, in order to tamp conveniently. ¹

Note 1.p.93. Naturally one wall of the form may be completed to its entire height, then the steel skeleton be set -- and as for piers -- the second wall of the form be built as the work advances.

For long walls, the forms may also be made portable, according to Sketch 42 b.

For particularly high walls a support from the ground would be too dear as a rule; then the already completed lower portion of the wall may be utilized for fastening the forms for the upper part, as in Fig. 42 c. The form surfaces are held together by bolts; for the surfaces may even be employed loops of annealed steel wire about 3 mm diameter as in Fig. 43. After removing the forms, the wire can be cut off short. The remainder and the bit of wood remains in the concrete. The fastening of the form surfaces in the lower finished portion of the concrete is best by bolts passing through imbedded concrete tubes or hollow tiles, then drawn out and used again in other places. The holes are later filled with concrete.

D. Means for making Concrete Watertight.

The concrete is not perfectly watertight; it contains pores, that are connected together, and that must first be closed, in order to make the concrete more suitable in practice. The concrete is more dense, the greater the content of cement, the more carefully the mixing proportions are determined, the greater the tamping, and the strength obtained, and the older *

the concrete. The mass is to be mixed fully wet; a small excess of * water is much less injurious than too little. The structure of the concrete becomes ever denser in time; concrete is not watertight at first, if it permanently comes in contact with water, it may become gradually watertight. It must have the fewest crevices possible, but when the mixture is too rich, is easily inclined to shrink, and is then dear on account of the high cement content. Experiments have shown, that gravel as aggregate produces more favorable results than crushed stone, but that too much water reduces its density.

A particularly careful smoothing of the surfaces wet with water requires great labor cost, and seldom affords the desired result,. Particularly at the corners of water tanks will the desired density by smoothing only be produced in time.

If special precautions are taken to produce a perfect density, the following points are to be considered:--

- a. The means must close the pores of the concrete with certainty and make them tight.
- b. The procedure must be readily checked and watched.
- c. The means must afford permanent protection from any kind of water.
- d. Loss of the means by mechanical abrasion or by elastic changes of form in the whole, that have as a result breaks or cracks, must be prevented.
- e. The means in a given case must afford protection against the chemical effects of acid water or steam.
- f. The means must be free from injurious materials, and affect the strength of the concrete as little as possible.

The use of special cements, for example of Star cement, with water resisting properties (addition of resinous substances), as well as the use of additions in making the concrete are generally less common than the use of plastering and coatings. * For reinforced concrete only plastering and coatings come into consideration -- or for high water pressure -- special tight facings.

1. Use of added Materials in making the Concrete.

Various means, representing potash soaps or tar mixtures, are added to the mixing water, thus giving to the concrete watertight properties. The same end is attained by hydraulic cementing substances (trass, lime). Materials of acid nature na-

nature have a decomposing effect. Concrete with a properly selected addition also then remains still watertight, if injuries of the outer surface result from mechanical abrasion (of running water containing sand, for example). In the preparation of the concrete is completed the work of making it tight. Disadvantages: -- generally reduction of strength of the concrete, since the additions may more or less hinder the necessary firm cementing together of the different materials; retarded setting; no protection from chemical injuries (only intended for chemically bad water); bad control in use (not always a proper distribution of the added materials); mostly unsuitable for reinforced concrete. The main thing in every case is a thoroughly approximate making of the concrete; defects in that cannot be compensated by the addition of materials to make it tight.

As preferable additions have proved to be hydraulic limes (or also Roman cements). If one substitutes in concrete for 1 part portland cement this material, then its tightness is increased in an important measure. For a rich mixture is to be taken less, for lean ones more addition of lime. The mixture of 1 cement + $1/2$ lime + 3 sand already proves to be watertight after 6 days. The more sand, the more lime paste is to be taken; thus the richer the mixture, the less the addition (addition about in proportion of $1/2$ to 2 parts by weight; to 1 part by weight of cement). The strength is little changed thereby, less in any case than by the use of fat lime, which is therefore never to be recommended. But if the strength be not so very important, then can concrete be made practically entirely watertight by the substitution of fat lime for half the Portland cement.

Favorable is an addition of trass and lime. ¹ Trass alone causes delay in hardening; but lime water favors the hardening of the mixture prepared with trass. The base of trass is the tufa stone formed of ejected volcanic trachyte masses, that occur in Germany chiefly in the province of Vorder Rhel (Netze valley). ²

Note 1.p.86. See Contribe. of Royal Testing Laboratory, Berlin-Gross-Lichterfelde. # 1313. Heft 2, as well as Zeits. für Tiefbau, 1914. p. 132.

Note 2.p.86. The particular peculiarities of trass-tufa are a definite content of included silicic acid and hydrated water,

as well as light volume with the tufo finely ground (to "trass").
 -- Trass cement mortar attains greater strength with increasing age than trass lime mortar.

A cheap method, but to be employed with care, is the addition of grease soap (potash soap made with good rapeseed oil).
 but no ordinary washing soap, and no alkaline grease soap! ¹
 The soap must be completely dissolved for use, best in warm water. One takes for 100 litres of water 4 to 5 kilos of soap; cement about 400 kil/m³ for concrete, sizes of aggregate up to 1.5 cm. Also advantageous is a smooth plastering about 1 cm thick composed of 3 fine sand, 1 cement, and water with the same content of soap. One m³ of concrete is about 8 to 10 marks dearer than without this addition. The greater the addition of soap, the more watertight, but so much the more is the reduction of the strength of the concrete (up to 50 p.c.). Likewise the setting period is shortened, so that its use in summer appears questionable. Concerning the presence of the watertight property, extended experiments do not exist. ²

Note 1.p.97. Only neutral potash soaps free from acid can be considered. Alkalies destroy the concrete. -- Moreover to be recommended is an addition of trass at the same time, since the soap alone as a rule lessens the capacity of the mixture for hardening.

Note 2.p.97. Thus special care is necessary, indeed both for the mineral oils as well as for the oil from petroleum residues.

Instead of the soap may be taken an addition of oil, about 5 to 10 p.c. of the cement contained. Benefits and disadvantages are about the same. ³ According to Page, plastering 15 to 20 mm thick makes it entirely watertight. ⁴

Note 3.p.97. See Cent. d. Boup. 1912. Nos. 11, 14, 15, 28.

Note 4.p.97. See B & M. 1911. p. 15, 348; 1912. p. 365. --
 Ann. B. 1912. p. 14, 241.

Eiber (Cornel Esser, Köln-Ehrenfeld) essentially consists of bituminous materials; it is added to the completed mortar. It is not combustible, and has proved itself in very many cases an excellent material for making concrete watertight. ⁵

Note 5.p.97. The materials for making watertight and for coating mentioned in this and the next Section have proved themselves as actually usable in practice. The composition of the materials is naturally a secret of the manufactory; by direct

inquiry of the Co. may be obtained accurate information concerning cost and use. There is a number of other authentic means, but which are similar in nature to those mentioned here.

Aquabar ("Aquabar G.m.b.H., Berlin), a paste composed of organic and inorganic substances, is packed in boxes. It is dissolved in water, and the concrete or cement mortar is made with this solution in the usual way. The cost of 1 m³ of concrete 1:3:5 is increased by this addition only 1 to 1.5 marks. The property of aquabar to intimately unite with the components of the water is based on the great content of included silicic acid, that crystallizes and forms fixed combinations. It forms an entirely safe protection from dampness and ground water. The strength of the concrete is reduced but little by the addition of aquabar, at most up to 5 p.c.

Preolite addition for mortar (A. Free, Dresden) is added to the water for mixing or to the mixture of cement and sand. There is needed for 1 m³ concrete or mortar 20 to 25 kil addition (supplied in form of paste or powder). The Preolite addition for mortar does not color the plastering, also has no injurious effect on the tensile and compressible strength of the concrete, and also does not delay the setting. ¹

Note 1.p.98. A similar addition is Ceresite.

2. Plastering for Waterproofing.

Plastering can be applied immediately after the completion of the structure (painting only after complete drying). But one waits for any settlement of the structure; plastering is never done in freezing weather, and in summer must be protected from sunshine. The greater the content of cement, so much the more effective is the plastering against vibrations; if necessary the plastering may be secured by fine wire netting. In plastering the corners of rectangular tanks, especial care naturally is to be taken. Disadvantageous for all plastering is the possibility of the formation of hair cracks, which by the effect of acid water may result in the destruction of the entire layer of plaster. As a protection against mechanical injuries is employed for floors special coatings of concrete or stone paving, for walls covering with glazed tiles or the like.

Covering with cement plastering (smooth coating) without any addition, after a previous cleaning and moistening of the wall. Mixture 1:1 to 1:2 (To 1 m³ sand about 700 to 800 kil cement). ²

Thickness of the layer at least 2 cm. The surface to be smoothed plain with the rubbing board or steel trowel (rubbed). Neat cement can only be employed for the entire layer, when the plastering is permanently under water or remains in closed rooms, thus not being exposed to sunshine or air currents. The plastering must first be completely hardened, before it comes into use.

Note 2.p.98. The plastering mortar must be made as fresh as possible. Likewise the surface in question must be plastered without interruptions as nearly as possible.

Use of a cement plastering with additions at the same time for protection from chemical injuries.

Biberol (Cornel Esser, Cöln-Ehrenfeld) is a paste addition without color or odor for plastering, which does not crack, & prevents flaking, and each coat after drying in air, receives oil colors and varnishes. Biberol is added to the water for the mortar. There is required for 1 m² external plastering 1.5 cm thick about 1/6 kil, and for 1 m² cellar plastering 2.5 cm thick about 1/4 kil. Cost at Cologne 0.80 to 0.90 mark per kil.

^{addition}
~~Awa~~ (A.W. Andernach, Beuel-o-Rh.) is a gelatinous mass without color or odor. In consequence of its antiseptic effect Awa also protects against formation of mould and fungus. It possesses a very strong resistance to water. According to certificate of the Royal Materials Testing Laboratory in Gross-Lichterfelde, Awa resists perfectly a water pressure of 6.84 atmospheres.

Lime and trass addition, 1 cement + 1 lime + 2 to 3 sand, = not too coarse and of mixed sizes of grains. A mixture 1:1:1 has a reduced strength as well as too long setting and hardening periods.

For small tanks of subordinate importance suffices a coating of the internal surfaces of the walls with clay or milk of cement. ¹ Also see B & E. 1912. p. 393.

Note 1.p.99. The cement powder is scattered over the water with constant stirring. The coating material is to be rapidly used with further continued stirring, since otherwise the cement sets and hardens on the surface. The surfaces to be treated are previously to be thoroughly wetted.

3. Coatings for Waterproofing.

There are employed preparations of tar and of soap, that fill the pores of the surface with a waterproofing sediment. They

present the following advantages:-- possibility of checking the work afterwards by the appearance, avoidance of hair cracks, protection from the effect of water and from chemical effects,² as well as from water in mortar containing sand. It is disadvantageous, that the concrete surfaces must be entirely dry at the application of the coating, so that one must wait until the concrete has entirely hardened and dried. Naturally the coating must not be injured at any place, since otherwise the moisture would gain admission to the unprotected concrete. Corners are always to be avoided or rounded. For the usual asphalt coating is required at least two applications; the second coat must only be given after the complete drying of the first one (which often takes several days).

Note 2.p.99. One is actually never entirely safe from the effect of acid water.

One of the best painting materials is the *Inertol* of Paul Lechler Co, Stuttgart. It is a paint ready for use, approved for several years, which serves to waterproof cement plastering and to preserve concrete structures, endangered by water, dampness, dilute acids and the like. Inertol not only forms a surface skin but penetrates into the cement and concrete, so that a coating of inertol can scarcely be rubbed off. Fully dried inertol has no odor, and gives no taste to drinking water.¹ Inertol has a much higher flame point than turpentine oil, for example, and paints containing benzole or turpentine oil. Inertol is prepared cold and dries very rapidly. The spread of it is considerable; for twice coating 100 m² of cement surface is reckoned about 30 kil inertol. (A cask containing 200 kil costs at Worms 75 marks per 100 kil). According to thorough experiments at the Materials Testing Laboratory of the Royal Polytechnic School in Stuttgart, the formation of shrinkage cracks in cement and concrete can be lessened; existing shrinkage cracks can be permanently covered and made tight by coatings of inertol.

Note 1.p.100. Inertol has been very commonly employed in tanks for drinking water and elsewhere. Any effect of taste or odor of the water by dried inertol has not yet been observed. Further see Cent d. Bauw. 1906, No. 75; 1912, No. 38; also Deuts. Bauz. 1908, No. 24; B & Z. 1912. p. 301.

Black Siderosthen-Lubross (Johannes Jeserich Co, Charlotten-

berg-Berlin) is a protective coating of especially great spread, and is applied cold. With 1 kil may be covered at the first coating 4 to 10 m², according to the nature of the surface to be covered; correspondingly more with the second coat. This covers the mass with an elastic coating like rubber, which is extremely resistant to atmospheric influences, as well as to the injurious effects of acid and alkaline waters, to free carbonic acid in water, rotten moss, etc. The skin coating produced by it is not affected by temperature changes in consequence of its high elasticity.

p/91. Preolite (A. Free, Dresden) is a black paint free from tar, of a thick fluid varnish nature. It is delivered ready for use, unites firmly with the concrete and is entirely resistant to acids and alkalis. At a test in the Royal Mechanical-Technical Experimental Laboratory in Dresden under water pressure of 12 atmospheres (water column of 120 m), there appeared neither admission of water or moisture into the test body.

Note 1. p. 101. See opinion of Prof. Dr. Rohland, Stuttgart. Contribs. 1908. p. 64; also B & E. 1914, p. 241.

Nigrite (Rosenzweig & Baumann, Cassel),¹ applied twice, soaks into the pores of the concrete and forms a very usable paint against dilute acids, ammonia water, carbonic acid water, etc. Also under the influence of fumes of muriatic acid, nigrite shows no alterations; it even exhibits a great resistance to the well known strongly effective acetic acid. Flaking of nigrite from cement does not occur.

Finally, attention should be called to a procedure especially favored in America, the so-called Sylvester process. This consists in preparing a 5 p.c. solution of alum and a 7 p.c. solution of soap; the soap solution is then applied with a soft whitewash brush to the clean surfaces. After the coat has dried somewhat it is brushed with clean water to remove from the surface the superfluous soap solution, and then follows the final coating with the alum solution.

On oil painting, see p. 113. On Murolineum paints (Droege & Fischer, Berlin-Friedenau), see Part II. 7th edit. p. 95.

4. Use of Covering Materials in Form of Slabs.

They serve for protection against the penetration of ground water and as waterproof coverings of bridges, passages, tunnels, etc. In every case they must be sufficiently elastic, to part-

participate in slight movements of the structure without danger. In consideration come tar-felt, asphalt layers, saturated fabrics (felt, jute, cloth, pasteboard or the like) with flexible protecting coats of asphalt on both sides, sheet lead with asphalt coatings. ^{p/98} 1 In regard to the arrangement of such protecting layers for insulating cellars, also see Part II, 7th edit. p. 158. 2

Note 1. p.102. Lead alone is subject to destruction by lime or cement.

Note 2. p.102. Also see Schultze, Waterproofing against Ground Water. 1912. Em. Ernst & Son. Berlin.

B. Precautions against Chemical and Electrolytical Effects. Danger from Lightning.

Unprotected concrete does not resist acids. ^{As} protection comes into consideration ~~the added and painting~~ ^{the added and painting} materials mentioned on p. 95 and 93, besides facings of glazed clay tiles, glass, porcelain and slate slabs, slabs proof against acids (Chamotte stone) and pottery slabs (Knauff's slabs). The joints must be as fine as possible and be filled with good acid proof cement. (For example with a mixture of red lead and glycerine combined as a cement). The main point is a smoothly executed covering of the cement surfaces.

Acid fumes are not injurious; yet also here is sometimes to be recommended a protecting coating.

Following are described various materials in regard to their chemical effect on concrete and the reverse.

1. Acids.

Very injurious are the strong acids, particularly such as combine with the lime of the cement in soluble lime salts; muriatic acid, acetic acid.

Injurious are further such acids that give with lime insoluble or but difficultly soluble combinations; there is here formed a protecting coating; sulphuric acid, fluoric acid, sulphurous acid. 3

Note 3. p.102. See B & E. 1914. p. 241.

Injurious is also tannic acid; very injurious is lactic acid. (For example, it is formed in fermenting sauerkraut; cement wash in dairies).

Very injurious is free carbonic acid dissolved in water.

As a permissible means of protection against very dilute ac-

acids (about 1 p.c.) is to be regarded Nigrite. ¹ Results with sulfuric acid, acetic acids, etc.). Otherwise a covering with tiles (Mettlach tiles, glass plates, etc.); also lining with sheet lead (dear). Coating with tar affords no permanent protection.

Note 1. p.103. See p. 101. Another means is concrete Murotin-
ous; see Part II, 7th edition, p. 95.

2. Fermenting Fluids.

They have an injurious effect by the formation of carbonic acid; saturation of the surface by tartaric acid or potassium-silico carbonate.

Very injurious is fermenting beer (also because acid) with tannic and carbonic acids. ² The means for protection here must not disturb the process of fermentation and neither yield taste nor smell. (Coatings with fluids containing silicic acid or a solution of cream of tartar; Heralite of Rosenzweig & Baumann Co., Cassel; a waxy coating of paraffine ³ after complete hardening of the concrete; lining with acid proof plates or clay tiles; covering with sheet aluminium is very advantageous).

Note 2. Under fermented beer with less than 0.1 p.c. acid content may already be injurious.

Note 3. p.103. But Resin-Paraffine coatings have also frequently had an injurious effect on the deposit and remaining of yeast in reinforced concrete tanks. See E & E. 1914. p.250.

3. Tars, Oils.

Injurious here is the possibility of forming acids; the more sebacic acid contained, the greater the injurious effect.

Tar exhibits no unfavorable influence.

Mineral oils are but slightly injurious, but under some conditions produce a slight reduction of strength. (Petroleum, ⁴ crude oil, naphtha, vulcan oil). Good plastering with cement coating is mostly sufficient.

Note 4. p.103. See E & E. 1914. p. 250.

Injurious are also the fatty oils, indeed on account of the formation of free sebacic acid by transformation. Animal oils; tallow, train oil, neatsfoot oil; vegetable oils; rapeseed, ¹ olive, ² castor, sesame oils. As protection; dense concrete, good plastering, painting (fluates), covering with tiles; but no asphalt.

Note 1. p.104. Concrete foundations of machines are gradually

gradually destroyed by continually dropping rapeseed oil and tallow.

4. Sewage, City Sewage, Manufactory Sewage.

Sewage in itself does not affect concrete. Carbonic acid in sewer water and also in sewer air (2 to 5 p.c.) does not attack the concrete. Likewise dilute acids (up to 0.1 p.c.) are not injurious. ³

Note 3.p.104. Fatty and slimy matters found in some waters deposit on the walls the so-called sewer coating, which affords a good protection. In any case, experience has taught, that a concrete is well preserved after long years for sewer works of all kinds. For water containing carbonic acid a quite dense mixture is to be taken for the concrete.

Injurious are concentrated acids (for example at places of the entrance of more acid water from manufactories). The effect is still more injurious with mechanical friction of the sediment. (Lines with strong fall and sliding sand). Protecting means; coating with asphalt varnish applied hot, etc.; lining with glazed tiles, construction with hard burned clinkers, cast acid proof asphalt joints with end joints in asphalt (for pipes).

Requiring no thought is the sewage from dyeworks, tanneries, bleacheries, paper, sugar, oil and fat manufactories, cloth and cotton factories, starch factories. (All wastes with alkaline and neutral reactions). Finally unnecessary is the consideration also of breweries and distilleries.

On the other hand are injuries of wastes from sulphuric and muriatic acids factories, from factories of artificial fertilizers, from tinning works, brass foundries, from ammonia and chloride of lime manufactories, coal mines, etc.

5. Mineral and Thermal Waters, Alkalies, Salt.

The waters are not injurious, if they are saturated with salts, that do not attack the cement. (Only injurious if hotter than 50° C. For mineral waters the molecular contained carbonic acid may be dangerous. -- Cement pipes for pipes from thermal springs require a protective coating. Springs containing common salt are not injurious.

Pure alkalies are without injurious influences (potash, soda, ammonia). -- Injurious in spring water with magnesium sulphate (bitter water). -- Carbonates of alkalies are not injurious, &

even increasing the strength of the concrete; soda (sodium carbonate) potassium carbonate, ammonium carbonate.

Neither injurious are the haloid salts of the alkalies; potassium chloride, common salt (sodium chloride), ¹ sal ammoniac (ammonium chloride). -- Calcium chloride and salpêtre (potassium or sodium ^{nitrate} chloride) are likewise not injurious. Magnesium chloride injuriously affects the hardening. -- Injurious effects have the sulphate salts soluble in water (potassium, sodium, calcium, iron and magnesium sulphates (bitter water).

Note 1.p.105. According to experiments of Tetmajer even strong solutions of common salt but slightly reduce the compressible strength in comparison with the water generally used.

6. Hot Water, Steam.

Hot water above 50° C has an injurious effect, and similarly also hot steam. That is particularly true for tanks in cold external air (great difference of temperature between external and internal surfaces). Previous cooling of the water in pipes; or double walls and heating the interspace before filling the hot water tank. ²

Note 2.p.105. In America artificial blocks of cement are often treated with steam under pressure in order to season them earlier, and to place them on sale. Experiments have shown, that steam pressure up to 5.6 kil/cm² accelerates the hardening and increased the strength. Am. B. 1913. p. 347.

7. Pure Water, Water containing Carbonic Acid.

Generally pure water is not injurious, assuming good plastering. But already water with little mineral components in solution may be injurious, when flowing permanently through concrete tanks. (Water reservoirs, settling basins).

Most injurious is carbonic acid in solution in water; protection; plastering 1:1, faced with neat cement, and painting with inertol, siderosthen, or the like.

8. Ground and Bog Water.

Ground water is injurious if it contains nitrate salts or chemicals, for example those serving to purify water for boilers or for other purposes.

Humic acid delays the hardening of the concrete.

Bog water is injurious, since besides humic acid it frequently contains also iron sulphate and free sulphuric acid (ground containing pyrites). ¹ Important for foundations and sewers;

a previous chemical examination of the ground and water is absolutely necessary before the construction.

Note 1.p.106. In Osnaburg a tamped concrete sewer was entirely destroyed by bog earth containing pyrites.

9. Sea Water.

Sea Water may have an injurious effect, indeed chiefly from the magnesium chloride and sulphate contained in it. These salts may cause a reduced strength of the concrete hardened in sea water.

Unfavorable is the uniform beat of the waves as well as the alternation of ebb and flow; the concrete is alternately wet and dry. (Constant change in volume).

But recent experiments and observations have shown, that the effect of sea water is scarcely of importance, if it concerns good and well tamped concrete with a rich mixture for the external surface.

Addition of brass has but a limited value. On the other hand, see Essay of Dr. Hambloch in Arm. b. 1914. p. 23.

10. Smoke Gases.

The injury from smoke gases ¹ for concrete masses (railway bridges, chimneys, locomotive sheds, heating pipes, etc.) is without importance; experiments have shown with experience, that the gases penetrate little into the concrete.

Note 1.p.107. Smoke gases chiefly consist of a strongly heated mixture of air with steam, sulphurous acid and soot.

Carbonic acid gas is not injurious, but even aids the hardening of the concrete. Only gases containing sulphur can affect concrete injuriously.

11. Metals.

Lead is destroyed by cement, especially in porous concrete. lead pipes are painted with asphalt varnish.

Zinc may also be attacked by cement. As protection is recommended painting with asphalt or a thin coating of lime paste.

Electrolytic Influences. (Danger from Lightning).

Concrete in a dry condition is a bad conductor of electric currents from machines, lightning and weak currents. On the contrary, if it be surrounded by earth, whose moisture is transferred to the concrete, then it will act as an electrical conductor. Only in such thoroughly damp reinforced concrete can occur injurious effects of electric currents (nothing is to

be feared in old and dry concrete); if the damp reinforced concrete is permanently traversed by constant current electricity, ² then first occurs an electrolytic decomposition of the water ¹ and a rusting of the reinforcement. Tamped concrete without reinforcement is not injured by electricity in dry or wet condition.

Note 2.p.107. With alternating currents, an electrolytic effect cannot occur.

Note 1.p.108. A very small addition of a salt (soda, common salt) increases the appearance of destruction an quite important measure.

The rails of an electric railway bedded in reinforced concrete are ^{permanently} exposed to wandering (return) currents, as well as to such currents that leave the proper conductors, and produce electrolytic effects in the neighboring pipes (water and gas mains) in combination with the dampness of the ground. These weak currents, but which flow for a long time regularly through the reinforced concrete, have effects far more injurious than currents of higher tension and voltage, that only occur for a brief time. The wandering currents are only without danger as long as the concrete is dry.

The best means for protection against the danger of electrolysis (formation of rust and bursting of the concrete) is a good insulation of the reinforced concrete from dampness by protection with asphalt, tar coatings, or the like, a good insulation of all electrical conductors, particularly of isolated beams embedded in concrete, and good insulation of all piping at the entrance and exit. Besides care must be taken to have a sufficient covering of the reinforcement; with careful execution a layer 2 cm thick suffices.

Genler, Dresden, established the following by experiments. ²

Note 2.p.108. B & E. 1910. Heft 11, 12.

1. For tamped concrete without reinforcement no notable reduction of the compressile resistance is to be expected from the effect of the electric current.

2. For large steel anodes embedded in concrete occurs rusting, if the steel forms the positive electrode, so that bursting of the enclosing concrete mass may occur.

3. For ordinary reinforced concrete structures according to results so far known, no anxiety occurs concerning the effect

of the electric current.

Brandt, Darmstadt, made thorough experiments ¹ with reinforced concrete cubes of 30 cm sides, that were exposed to the various effects of the electric current. For a number of cubes the reinforcement was exposed for about 6 hours daily for two years to a current of 140 volts and at least 6 amperes; neither did it show the formation of rust nor was the concrete softened. Further experiments led to the judgment, that at all such places where the concrete is in damp or wet condition and is not sufficiently insulated, formation of rust occurs on the reinforcement -- and in consequence of expansion by rust -- bursting appears in time, as soon as this concrete is exposed to an electrolytic current (constant current). ² The assumption appears entirely justified, that for well dried reinforced concrete structures in dry air no danger of any kind exists for destruction by electrolytic influences. ³ Care was here taken, that the current used in the experiments was 70 times as strong as those wandering earth currents could be.

Further see B & E. 1912. p. 223; 1913, p. 132, 290, 306, 331, 445. -- Cent d. Bauv. 1911. No. 75; 1913. No. 80. -- Arm. B. 1911. p. 350; 1911, p. 32; 1912, p. 32; 1913, p. 166.

Note 1. p. 109. Heft 8 of *Ann. Chem. Phys.*; also see B & E. 1912, Supplement Heft.

Note 2. p. 109. Naturally for buildings the reinforcements must not by fastenings be placed in direct connection with the electric conductors of strong currents.

Note 3. p. 109. The best protection is formed by a coating of resin. Mixtures of soap, ceresite or the like (p. 98) for making concrete watertight are not preserved under electrolytic influences.

Lightning strokes produce only local disturbances in a reinforced concrete structure, where they must pass through badly conducting masses. (Concrete without reinforcement). But usually the stroke is carried off by the steel skeleton embedded in the concrete in a sufficiently permissible manner; slight injuries to the concrete covering at the places struck, and at places where the lightning must spring from one iron rod to another within the concrete masses means little. ¹ A special protection from lightning (copper rods) is not necessary; but it is advisable to connect the steel reinforcements by wires eve-

everywhere, likewise the gas and water pipes, as well as the roof gutters and the leaders. It is likewise recommended to join the reinforcements below to the conducting ground. ² To catch the stroke (according to ^{Ruppel} Ruppel) round or flat bars can be employed at distances of 5 to 10 m, which are connected by wiring with the reinforcement of the roof slab, and need not extend more than about 25 cm above that part of the building. (ridge of roof). ³ No danger to life exists from a lightning stroke, since the conductor is enclosed entirely in the concrete.

Note 1. p. 110. There are formed the so-called metal tubes, which correspond to the "lightning tubes" found in sandy ground.

Note 2. p. 110. Always exists the danger then, that the ends in the earth may give admission to injurious currents, coming from the neighboring pipe lines.

Note 3. p. 110. One may also use ornamental finials on the gables, connected with the steel reinforcement. For flat roofs such arrangements naturally are unnecessary; the disseminated electricity is there distributed over the great area of the entire roof. -- As injurious may be regarded the fact, that with the connection of the receiving rod and the steel skeleton, even without a lightning stroke a permanent quiet passage of electricity may occur, whose constant influence should be prevented.

Very worthy of consideration are the experiments of Ruppel (Frankfurt-a-M.), in which the tension was about 100,000 volts, the current being about 3,000 amperes (corresponding to 300,000 kilowatts). ⁴ The experiments substantially confirmed the results of Berndt (Darmstadt). Ruppel showed, that the so-called equalizing or obviating effect of a lightning conductor for practice does not come in question (platinum and other points are entirely unnecessary), that the lightning seeks to pass from the place struck by the shortest possible path, -- following metal to the best earth point, and that all destructive effects in jumping across are the smaller, the more the lightning can branch from the point struck; for the more widely the discharge passes downward, the weaker is its effect. It can only be dangerous, where the current is concentrated. But by the numerous embedded steel rods is primarily afforded a very great possibility for the division of the lightning stroke. For a small structure with eight columns (each with 4 rods ^{P. 111} 20

20 mm diameter, for example, there was at command a circular section of about $10,000 \text{ mm}^2$, in contrast to $2 \times 50 = 100 \text{ mm}^2$ for an ordinary lightning conductor.

Note 4.v.110. See Contribs. 1913. p. 133; 1914. p. 43, 61, 71, 80. -- Further, Cent. d. Bauv. 1913. p. 529 (lightning stroke on a reinforced concrete mast, p. 528 (lightning stroke on a domed roof), Clay Industries, Zeits. 1911. No. 110.

Likewise the experiments made by Berndt in Darmstadt (p. 109) permit with sufficient safety the conclusion, that for reinforced concrete structures at least no greater damage from lightning exists than for buildings of any other kind, and that reinforced concrete construction as such does not have its bearing capacity endangered by any lightning stroke. ✓

F. Treatment of the visible Surfaces.

The gray color of the concrete has a monotonous and cold unpleasant effect, particularly when the great surfaces are concerned, which generally appear. If the surfaces are not to remain as they come from the ordinary (unplaned) forms, then the necessary animation can result in different ways.

a. Subdivision of the rough surface of the concrete by bands projecting some cm with separate panel surfaces. (Grain of wood and joints of forms remain visible).

b. Use of planed forms (smoother visible surfaces).

c. Cement plastering.

d. Cement plastering with later painting. (Fluates, light-colored Siderosthen-Lubrose, etc.).

e. Facing concrete, unworked, treated with acid, or wrought by stonecutter.

f. Facing concrete, wrought by stonecutter and painted.

g. Facing with ashlar, tiles and the like.

If the visible surfaces of the structure are yet to be plastered for sake of better appearance, this is to be commenced at once after removal of the centering. In times of frost naturally all plastering work -- excepting of enclosed rooms -- is to be discontinued, so far as no means of protection from frost is employed. (See Section IV d). In summer, care is to be taken for protection from direct sunshine, and to wet the plastering more frequently after setting. The thickness of the plastering is according to the roughness of the surface, but is not to exceed 1 or 2 cm. ¹ In all cases must the con-

concrete be roughed beforehand, cleaned well from dirt and flakes with wire brushes and carefully washed, when possible with the aid of a 20 p.c. solution of muriatic acid, which must then be removed by strong washing. Also advisable in a given case is a preliminary coating with thin cement milk. The cement plastering is to be firmly applied ² and then pressed with a rubbing board and smoothed, only when the concrete is completely dried and hardened. If the plastering is to serve later as ground for valuable paintings, it is frequently advisable before the application of the plaster to fasten a slight wire netting or expanded metal to the concrete. Finally also nails alone suffice, that are driven in great number and not like pins. ³ The plastering is to be protected from direct sunshine and kept constantly damp until it is completely set.

Note 1. p.112. To prevent any lack of clearness in the accounts, it is advisable to state in the estimate the cost:-- price per m² plastering as additional cost of concrete. Otherwise special attention must be called, that the thickness of the concrete given in the drawing is to be understood as without plastering, that thus the plastering is not to be paid for as an extra.

Note 2. p.112. Recently machines for roughcasting a wall are frequently employed. The arrangement consists of a mortar mixer, a compressed air machine, and a mortar pump, which are driven by a strong motor, the mortar being forced through a hose with nozzle under a pressure of 1 1/2 atmospheres, and thus the wall and ceiling is coated with plaster. If an hourly use of 2 to 3 m³ of mortar is reckoned, one can cover 120 to 170 m² of surface with plaster 1.5 cm thick. The plaster sets more rapidly and strongly than plaster applied by hand.

Note 3. p.112. Another method consists in dividing the entire surface by inserted thin strips into separate panels while concreting, these strips being then removed; the plastering can then adhere in the grooves thus formed.

#113 A good mixture for cement plastering is:-- 1 cement + 2 to 3 sharp sand, applied in two coats if possible, with the addition of some lime in the second coat. One may also use a true cement-lime mortar (1 cement + 1 lime + 5 to 6 sand) in case of twice coating with casein colors, fluates, etc. But in general lime mortar for plastering is less to be recommended, si-

~~not such plastering -- without fair~~

such plastering -- without painting -- in consequence of the porous surface resists weather less, and also easily becomes dirty.

The surface of the cement plastering is always alkaline, i. e., red litmus paper is colored blue, when pressed against the cement with a cork. If the surface of the cement plastering is coated with Kehler's Fluat until the surface reacts acid, i. e., until blue litmus paper reddens, and if finally one washes off the superfluous salt with water, then the plastering can be painted in oil colors. ¹ The oil color will then adhere to the plastering as well as on lime. ²

Note 1. p. 113. Oil colors must not be used on fresh cement plastering, since the cement contains certain acids, with which the oil colors do not agree. If one does not use fluorates, then is recommended a preliminary ground of hot linseed oil varnish. The oil color is to be applied in two or three thin coats on the dried linseed oil, each coat drying by itself. Also sometimes recommended is a preliminary treatment of the surface to be coated with very dilute sulphuric acid (1:100), a solution of zinc sulphate or one of ammonium carbonate.

Note 2. p. 113. Otherwise are to be used only earthy colors and some metallic colors, for example, cobalt blue, zinc white, ultramarine, barytes sulphate, bone-black, ochre, etc. On the combination of coloring materials with cement, see p. 25.

To produce a uniform effect in color a colored sand is preferred, like Ulm ground stone. Colored cements are to be treated with great prudence.

It is counted as a disadvantage to concrete ceilings, that the plastering adheres badly, and scales off too easily. The causes of this bad condition are chiefly the following:-- Smoothness or dirty surfaces of the concrete, plastering too rich, use of sand with particles of coal or unslaked lime, concrete still damp, dampness rising from the ground, plastering dried too rapidly, effect of frost, direct effect of wind and sunshine, etc. Particularly dampness forms the chief reason for formation of cracks in plastering. The water conducts the salts (mostly sodium sulphate or magnesia sulphate) to the external surface (efflorescence); the water evaporates, and the salts remaining expand the external layer of concrete by increasing its volume. The separation of the salts is best impeded by me-

of increasing the density. Likewise must sufficient protection be made against the dampness rising from the ground. (Isolation of walls). The plastering for the ceiling is made somewhat richer than for the walls, and if the ceiling can be previously covered by a light coat of cement mortar 1:3, the steel parts will be well covered then. Under some conditions to produce better adhesion of the plaster is recommended, first a thin layer of gravel on the centering for the ceiling, or of chips (granite), which then produces a very rough and spiky, nearly free surface for the adhering plaster.

On hair cracks, see p. 30; also Sections V B and XIII.

For buildings of minor importance or for those designed to be seen afar, and representing purely utilitarian structures, it generally suffices to leave the visible surfaces as they come from the forms. In this case naturally special emphasis is placed on careful centering, and only planed centering is used. The ridges formed by the jointing of the planks can be cut off or hammered off. If one wishes to do otherwise, then is recommended a later coating of the surface with cement milk (white) with a covering of yellowish-white lime paste or a yellowish-white paint blown on, or even rubbing with sharp wire brushes and dilute muriatic acid.

In America the following treatment of visible surfaces has been frequently adopted:-- the centering is removed after setting, but still before complete hardening of the concrete, and the visible surfaces are directly afterward washed with water. By washing is removed the thin coating of cement, that enclosed the grains of sand and gravel, so that these are laid bare. The appearance of the visible surfaces thus treated naturally depends on the kind of aggregate and its distribution in the concrete mass. Likewise a certain gradation is produced by the degree of washing. If properly done, then with the brush can be removed so much mortar, that a strongly effective surface is produced, which for buildings of monumental ^{appearance} ~~importance~~ and with bold forms is entirely in place. For light and pleasing buildings the effect of the surfaces must be correspondingly more graceful, which can be produced by the selection of small grained aggregate. Fig. 44 shows a washed and brushed concrete surface, consisting of 1 Portland cement + 2 pit sand + 3 sifted river sand with 9.5 mm grains. (The concrete in Fig. 45 is

Note 1. p.118. With the use of gravel sand for the building, it suffices under some conditions to remove the coarse grains or also the too fine sand from the facing concrete -- according to the intended effect of the surface. One also employs the so-called rippled concrete (for example, 1 cement + 1 sand + 2 pea gravel), which is dressed later.

Note 2. p.118. If the concrete of the reinforced concrete members be quite fine grained, then may one omit a special facing concrete and have the stonemason dress the surfaces, like those of the building.

Generally valid rules for mixing proportions of facing concrete naturally cannot be established. For use are particularly suitable crushed dolomite and basalt, white and gray granite, limestone and shell limestone, and some mixtures used with advantage are the following:--

- 1 cement + 1 sand + 3 fine crushed stone (chips).
- 1 cement + 1 basalt sand + 1.5 crushed basalt (strong gray color with black spots).
- 1 cement + 1.5 sand + 1.5 crushed limestone (light gray color).
- 1 cement + 3 ground granite (granite tone).
- 1 cement + 4 sand + 4 crushed porphyry + 1 Uln white (looks like lime-tufa).

For the placing of the facing concrete is best employed a vertical board or plate with handles, which corresponds in height to a single tamped layer, and according to Fig. 46 is placed at 5 to 12 cm from the external form. In this way a mixture of the facing concrete with the concrete nucleus behind it in tamping is prevented. After completing a layer of concrete, the facing concrete is placed between the separating plate and the external form to the height of a tamped layer, the plate is again drawn up and the entire layer is strongly tamped at the same time, including the facing concrete, so that a good junction between the two kinds of concrete is obtained, and a later displacement of the facing is not to be feared. If one desires to obtain a special animation of the visible surfaces, then may be sifted and specially suitable pieces of aggregate be inserted and pressed against the external form. In the later dressing of the visible surfaces, such stones advantageously appear.

The proper dressing follows by the stonemason with the hand

or compressed air bushhammer, ¹ and indeed after the complete hardening of the concrete. No bits of stone should break out, since then unsightly holes would occur. To produce a better effect the surfaces may be differently treated, partly bushhammered, partly drowed, etc.; certain portions may also be left uncut, thus remaining rough from the forms.

Note 1.p.117. Bushhammered, crandalled or pointed produce uniformly animated surfaces, drowing, fine or coarse striped surfaces. Since by crandalling the angles are easily broken off, dressed surfaces -- for better appearance -- are preferably produced with a tooth chisel (drafted margin). But generally the surfaces are bushhammered.

Recently the sand blast is also used for dressing, whose effect is to give the surfaces a uniformly rough appearance. The sand driven by compressed air certainly dresses the entire surface like a pointed hammer. ¹

Note 1.p.118. A sand blast apparatus consists of :--

1. The sand blast (the hose leading the sand stream may be up to 30 m in length.
2. The compressor for producing the compressed air (driven by electric or benzine motor, or by existing power.
3. The air reservoir for receiving the air.

If one has employed colored gravels for the facing, these can be washed with wire brushes and dilute acid, and allowed to appear. But the surface must be well washed with pure water. If color be mixed with the facing concrete, then may only be used mineral colors, that are not affected by alkalies. Better is the use of ground stone in place of sand.

For polishing afterwards a preliminary selection of aggregate is necessary under certain principles. The polishing is preceded by a careful rubbing, in a given case by an electrically driven easily portable grinding machine.

Instead of a dressed facing concrete, also facings of ashlars, split stone, of clay, marble or porcelain tiles, may be taken (for example, in bath houses, corridors, etc.). It is always advantageous, if the building material remains uncovered -- as in facings -- since otherwise the concrete cannot be utilized in its entire thickness for the supporting construction. Disadvantageous is the difference in the elasticity under varying stresses and changes of temperature.

For the Evangelical Garrison Church in Ulm the concrete surfaces of the interior are effectually animated by the inlaying of colored clay tiles in the hammered dark gray facing concrete (crushed basalt and fine pebbles). The tiles have holes through the pegs projecting on the back, through which were drawn fine wire anchors. The tiles were laid on the form, fastened in the proper place by wires, then tamped over. ¹

Note 1. p.119. Deuts. Bauz. 1910. p.226. -- To animate the visible surfaces gilded mosaic pieces were also tamped in.

Artistically executed ornamental slabs (for example ceiling coffers) can be made in plaster moulds, afterwards being set. Instead of being finished by a stonemason suffices a washing with acid. The underside of the dome of the National Battle Monument (with figure ornamentation) was concreted on a waste plaster mould, but not worked over afterwards, only improved as far as necessary.

Further see B & E, 1912, p.230 (Treatment of the concrete surfaces with mica), Artificial Stone Industry, 1912, Nos. 24, 25 (Colored ornamental procedure for visible surfaces of concrete), Contribs. 1911, p. 137, 147 (Architectural works with facing concrete), Industrial Architecture, 1914, Heft. VII, & VIII (concrete and ceramics). further see Cement & E. 1911, p. 453; also Patry, "Dressed concrete ashlars and artistic treatment of concrete." p. 175.

G. Testing of Concrete (Cubes and Beams). ²

Note 2. p.119. This treats of tests made before the commencement or during the construction. On later tests of completed structures, see Section IV f.

Building officials are justified in requiring a test of the strength of the concrete; for according to what has been stated on p. 97, the proportions of mixture do not alone suffice for deciding on the goodness of the concrete. Since now on the ground of the ministerial decree, the tensile resistance is only considered under certain conditions in the calculation of reinforced concrete structures, the testing first extends to obtaining the resistance to crushing. Every arrangement for making tests, that must serve for current supervision of the concrete work, must be durable and strong, even under rough treatment, must have small cost of construction, give sufficiently accurate values for experiments, be easily handled and portable;

required, as a rule is not indispensably necessary. Bach has been established by experiments, that shocks before the setting of the concrete increase its resistance. Cubes have been taken directly after making for a distance of 70 m. or a km., and it was found that the resistance of port became higher, than for cubes quietly viewed. In other experiments no difference was found between these blocks and those transported while fresh. (See transported concrete, p. 82).

Note 1, p. 121. Further data on making and treating test blocks are contained in "General Regulations for preparing, treating and testing Buildings of tamped Concrete" (issued by German Commission for Reinforced Concrete, 1908).

A. Standards for comparative experiments on tamped concrete, (Laboratory experiments).

B. Regulations for experiments on crushing during the erection of Buildings of tamped concrete.

If one desires to have an examination of concrete by a Materials Testing Laboratory, then either ^{ready} send test cubes can be sent, or the test cubes be made in the testing laboratory with the following statements.

1. Nature and composition of the mixture; amount of water added; mode of mixing the raw materials; mode of tamping, thickness of tamped layers, weight of tamper, duration of tamping.

2. Dimensions of test block.

3. Mixed in parts by volume or weight?

4. At what age is test made (7 days, 28 days, 3 months)?

5. How were the blocks hardened before testing (in air, watered 2 to 3 days, once daily, under water or in damp sand?

6. Tested in direction of tamping or perpendicular thereto?

The testing of concrete only 28 days old cannot show at all such a favorable result as if the concrete were 3 months old.

In general the resistance of concrete at 28 and at 90 days is in the ratio of 3 to 4 (with a rich mixture) or 2 to 3 (with a lean mixture). After a year the resistance is more than twice as great as after 28 days. Bach's experiments ("Resistance and Elasticity of Concrete with varied Additions of water") have shown, that an increase of resistance is still to be clearly noted after 6 years, and that only the compressible elasticity (coefficient of extension) is reduced by age. Bach found for concrete 6 years old, 1 cement + 2.5 sand + 2.25 fine gravel + 3 coarse crushed stone, a resistance of 530 kil/cm². with

regard to the actual conditions of practice, it is always to be greatly desired, that an age of 90 days be prescribed for test cubes, as the Regulations of the French government require for reinforced concrete structures, and as different technicians have recommended. Men indeed make the opposition, that after the course of three months, the entire building is often already completed; yet the almost necessary ^{improvements} ~~alterations~~ and strengthening for bad results, if not even in most cases, are just as impossible after 28 as after 90 days. If necessary -- with unsatisfactory results of testing -- can a somewhat later removal of the forms and scaffold timbers be required, and finally also a pointing of the concrete. But further it is to be considered, that when the computed live load comes into effect, the concrete is considerably older than 28 days. If one does not wish to delay the beginning of the building, then he may be satisfied by a preliminary crushing test after 14 days, which is later followed by the final test after 90 days. (In Hamburg for large buildings usually 2 to 4 series of experiments with 5 cubes in each are required, corresponding to the different parts of the structure. Indeed 2 cubes are crushed after 7 days, 2 after 28, and 1 after 45 or 90 days.

Note 1. p. 122. As an average value for the ratio of strength of a 7 day cube to that of 28 days may be taken 2:3. Am. B. 1918. p. 328.

A test of the cube before the bid, as an addition to this, in a given case can only be required; if the purpose exist for introducing higher permissible stresses (50 to 70 kil/cm²) in the calculations, for example for compressed parts with very close set stirrups (and arches between beams, abutments, etc.). Otherwise the requirement of tests of resistance to crushing before the contract would be quite oppressive to the contractor, the work causing a useless expense, at the same time that the contractor must make far more estimates, than he can generally undertake.

The crushing results of different cubes often vary very materially, wherefore as a rule the average of three tests of cubes is taken. Such diversity is primarily due to the mode of making the cubes, exact rules for the proper tamping indeed exist, yet one is very dependent on the skill of the workmen. A thoroughly uniform tamping is very difficult, and therefore ~~one of the tests of cubes may often be as~~

value of cubes tests may often be of a quite questionable nature. The height of fall of the tamper (25 to 30 cm) is hard to retain throughout, quite aside from the fact, that one also does not always have to do with a free fall of the tamper. In any case it is not well possible to harmonize the tamping at the building with that on the cube, particularly when steel reinforcement is there used, since the use of ordinary tampers is not generally admissible. The usual dimensions of cubes are with 30 cm sides. Smaller cubes (of 20 cm or 10 cm) show greater resistance values.¹ Finally, it should be mentioned that the test cubes should not be made in cold weather, since during the effect of freezing naturally occurs but a slight increase in the resistance of the concrete, and it has indeed been observed, that the effect of freezing on the lean mixture is worse, as a rule, than for richer mixtures.

Note 1. p. 123. The Swiss Regulations require a series of 3 cubes of 16 cm or prisms of $36 \times 12 \times 12$ cm; the latter may also be used for determining resistance to tension by bending test. -- For smaller cubes must also be used smaller grains, wherefore they cannot always be termed suitable, in spite of easier and more rapid preparation.

The resistance to crushing diminishes with the increasing dimensions of the cube. (See Arm. B. 1912. p. 288).

As determinative resistances to compression according to the "General Regulations" (1908) for the construction and testing of buildings of tamped concrete are to be taken the mean values of the resistance values by the maximum height of the manometer. The actual height of the manometer doubtless gives a much clearer result for determining the resistance of any sort of concrete, than the beginning of the formation of the first crack, (Württemberg Railway Regulations, Dec. 1912), since one at first only has to do here with the unimportant flaking or splitting of the angles, first occasioned by the irregularities of the previously rough surfaces of the cube; it is therefore also difficult to determine without error the exact beginning of the formation of a crack. In these compression tests in laboratories for testing materials, the rough surface of the cube is previously made smooth with cement or cement mortar, and then ground later. The cubes tamped at the building with on the contrary mostly come in rough and unprepared condition under the

artens' press at the manufactory, so that already from beforehand -- if one desires to determine the resistance to crushing on the basis of the formation of the first crack -- an unfavorable result is to be expected. The cubes are to be compressed until the pressure indicator of the testing machine shows a $\pm$ loss of pressure, then until the resistance of the compressed body is finally overcome.¹

Note 1. p. 124. Also Bach understands by resistance to crushing, that pressure which corresponds to the largest loading, the so-called breaking load. (Bach, Theory of Elasticity, p. 127 of 2nd edition).

One sometimes hears the opinion, that the resistance of a test cube is greater than the resistance of the concrete in the building itself. But experience has proved, that the resistance in the structure may generally be quite considerably greater than the resistance of the cube. In this respect, reference is made to Regulation 5 for crushing experiments on the construction of buildings of reinforced concrete, that was made by the German Commission on Reinforced Concrete. That states among other things: -- "One considers, that according to former experiments a concrete mass with once a rich addition of water, as must frequently be employed in reinforced concrete structures, in consequence of its preparation in the tight iron moulds impermeable by water, has a smaller resistance than the same concrete mass, which loses water through the wooden forms and the joints contained therein." For the test cubes also the conditions are unfavorable, since already on account of the great differences of the mass, the compacting of the concrete may be much less complete, than on the structure. Soft concrete may still be tamped well on other concrete, but badly in iron forms. It is also to be taken into account, that with the iron forms the water contained in the concrete cannot be distributed in the mass, as is the case in structural members in the wooden forms. For also the variable depositing of the concrete -- sometimes with and sometimes without pressure by the structural members -- as well as to take into account the circumstance, that the beginning of the hardening of the concrete (within the first 24 hours) is varied, and indeed in consequence of the unequal condition of the mass in setting in the large and small masses. And finally is to be considered

also still the impossibility of obtaining a similar hardening procedure in the cube and in the structure, since the magnitude of the upper surface naturally is not without influence on the hardening procedure. Professor Möller, Brunswick, states, that in his experiments with T-beams, the safety in the building was nearly 50 p.c. greater, than was determined by the results of the crushing tests of cubes. ¹ The making of test cubes should indeed serve first to afford apparent evidence, that the concrete was correctly made, by the use of the proper sand and the corresponding aggregate. One will have approximate data on the usability of the concrete itself. This treats more or less of the working test; tests of quality are more a matter of the laboratories, where especially comparative crushing tests are undertaken with the use of different kinds of aggregate and sand. Also there machines for mechanically tamping cubes are at hand, which make possible a perfectly uniform construction of the series of cubes. (Tamping machine of Schmidt) ² A destruction of a concrete structure by insufficient resistance to crushing comes in question only the rarest cases. ³

Note 1. p. 125. Besides it is to be considered, that the resistances of concrete in compression obtained by bending tests of T-beams are about 1.2 to 1.25, and for rectangular beams $\pm$ are about 1.4 times the resistance of a cube.

Note 2. p. 125. It has been found by experiments, that between the resistance of blocks made with the most exact tamping by hand and with the Schmidt machine no difference exists. According to the statement of the German Concrete Union and the Union of the Portland Cement Manufacturers, such blocks made with the new Schmidt machine are taken as if made according to the new official standards for comparative crushing tests with tamped concrete. The machine can be employed both for the determination of the actual resistance, as well as for obtaining the best mixing proportions and the best raw materials, indeed for cubes of 30, 20 and 10 cm sides. A cube of 30 cm is tamped with 218 blows in 8 minutes.

Note 3. p. 125. On tests of cubes, see further Section VII. 1. Following is reproduced the regular report of a crushing test as an example.

Report of testing on Jan. 19, 1914, in the Factory of the undesignated Firm, of crushing tests on large test cubes with the

mixture of 1:2.5:2.5 (concrete of crushed stone for piles and supporting beams), and three test cubes with mixture 1:3:5. (Gravel concrete for foundations).

Dimensions, $30 \times 30 \times 30$ cm.

Cubes 1:2.5:2.5, 52 days old; made Nov. 27, 1913.

Cubes 1:3:5, 39 days old; made Dec. 10, 1913.

Compression was applied perpendicular to direction of laying.

Tests were made on machine of German Concrete Union, Wartens system, and produced the following results.

	. mix. 1:2.5:2.5			. Mix. 1:3:5..		
No. of cube	I	II	III	IV	V	VI.
Wt. in kil.	62.8	63.0	63.7	65.0	65.5	65.5
Comp. at first crack in kil/cm ² 1	268	276	234	163	167	180
Comp. at highest manometer pressure in kil/cm ²	335	285	301	174	182	205

(For cube I of mixture 1:2.5:2.5 the crushing resistance was not determined on account of first reaching the maximum capacity of the machine.).

Note 1. v. 186. The values of this series might also be noted.

As determinate crushing resistances may be taken the average of values for crushing obtained by maximum heights of manometer, according to General Regulations for preparing, constructing and testing buildings of loaded concrete (1905), thus being 307 kil/cm^2 for mixture 1:2.5:2.5.

187 kil/cm^2 for mixture 1:3:5.

Dresden. Jan. 19. 1914.

(Subscribed).

2. Tests of Beams.

It is without doubt, that the method of testing cubes of concrete has many disadvantages and sources of error (use of heavy, costly, and difficultly portable hydraulic presses, frequently with not entirely reliable manometers; lack of mathematically plane surfaces, therefore no uniform distribution of pressure; care of transportation of cubes; frequent delays in making crushing tests in the laboratory). Compression tests with concrete beams generally give greater values for resistance than tests of cubes. In every case must sufficient steel be in the tension zone of the beam (about 4 p.c.). The results (according to experiments of Schüle) are the more regular, the

greater the dimensions are taken. The beam tests can properly be recommended only for the determination of rectangular beams, in any case also for slabs and T-beams, but least for columns. The defects of all beam tests are the following:-- greater liability of the concrete beams to injuries (by rail transportation); greater effect of size of grains and of tamping; possibility of slipping of reinforcement; indeterminate influence of size of reinforcement rods.

Note 1. p. 127. Since the regulations are all based on the resistance of cubes, experiments with some cubes at the same time must give the ^{value} ~~value~~ for calculations. According to the experiments of the German Commission on Reinforced Concrete (Heft 18), the ratio of the resistance to bending to resistance of a cube was determined as averaging 1.7. It was further determined, that the effect by the addition of water makes itself important; the ratio was 1.91 for earth-damp, 1.83 for soft, and 1.58 for wet mixture. The experiments showed in all cases, that the beam test may be regarded as a substitute for the cube test.

On tests of beams, see B & E. 1910, p. 149; 1911, p. 249, 284, 301, 405; 1912, p. 10, 212, 280. Arm. B. 1911, p. 117, 151, 231, 263; 1912, p. 286; 1913, p. 143. Further see Researches in domain of Reinforced Concrete. Heft 14. Emperger, "A Test of Quality of Concrete". Arm. B. 1913. p. 143. Zeits. d. Arch. u. Ing. Union. 1912. p. 177, 282. Contrivs. 1912. p. 55; 1914. p. 34. P. 128 F. von Emperger's Beam Tester (control beams) permits a rapid test of the quality at the building itself. Section and length of the beam are found in Figs. 47, 48. As loading in the maximum case 180 bricks (800 kil) came into consideration, piled in 10 courses, and the concrete beam is loaded at two points. The total cost of the necessary apparatus for the beam tester amounts to 250 marks. In this is not included the framework represented, which is easily constructed at each building site. (Chemical Laboratory for Clay Industries). Berlin).

The Reform Testing Machine, patent of Buchneim & Reister, Frankfurt-o-M. likewise presents a simple and rapid making of a large series of experiments; within 2 hours can be made 24 tests without waste. The apparatus itself is shown in its entirety in Fig. 49. The beam is tested vertically; the portion tested remains free from transverse forces. Bottom and top le-

levers are connected by a tension rod, whose length may be shortened by turning a hand wheel, but to be able to measure the stress at any time, the tension rod does not directly join the top lever, but by the insertion of a weighing lever, which reduces the stress in the tension rod to about $1/40$, shown at x the longer end. This hangs an aluminum vessel, that receives water as a weight; after opening a cock, it runs into a vessel connected with the bottom lever. Hence the increase in load on the weighing lever is thus quick and ~~sensitive~~ ^{continuous}. The operator has only to take care -- by turning the hand wheel, that the weighing lever hangs free. As soon as the test piece yields, the weighing lever falls and automatically stops the apparatus. By simply turning around the test piece, the same can first be tested for tension and then for compression. The machine is packed in a case ready for use; its weight amounts to about 180 lbs. Cost of the machine packed ready for transportation 1050 kls.

The test beams (according to Fig. 50) are 1 m long, 10 cm wide and 12 cm deep. Thus they are quite heavy. They are reinforced by flat bars, recovered after testing the piece., therefore can be used permanently. The flats are curved in semicircles at the ends, thus forming both top and bottom ends. Stirrups are autogenic welded.

The capacity of the Reform Testing Machine extends to about 430 kil/cm^2 resistance of the beam, which suffices for most practical purposes. The stress in the concrete corresponding to each load is found from Tables. ¹

Note 1. p.129. See B & E. 1911, p. 394; 1912, p. 28, 92, 252, 290. Contribute. 1912. p. 133.

SECTION IV. CONSTRUCTION OF THE BUILDING.

a. Arrangement of the Building Site.

First is concerned the most economical and most suitable arrangement of the building site, and a proper distribution of all works of construction, with corresponding consideration of the local ^{conditions} ~~considerations~~ of the ground. First must an uninterrupted procedure of the work be taken into consideration, several beginning places must be arranged for great building spaces, so that the divided working groups may not disturb each other, and each retain a sufficient freedom of movement. Building sheds are to be erected for the superintendence and the workmen, as well as for the building tools, cement, etc. ² Then are to be arranged places for storing scaffold timber, steel, sand, crushed stone and -- sheltered as much as possible, working places and sheds for carpenters and steel men, ¹ to lay transverse rails (narrow gauge) from the terminal rails to the building site; for the transportation of materials must be as cheap as possible. Finally, care is to be taken for water supply, for lighting the building site, and for the correct distribution of all machines. ² The mixing machine is to be so placed if possible, that -- according to the progress of the work -- it may require but small transportation of the concrete mass. To be considered also are dangers from fire. (See p. 77).

Note 2. p. 129. On the construction of a portable shed for cement, see Z. & B. 1903, p. 649.

Note 1. p. 130. If one has at command a well arranged factory, then the forms can be thus prepared beforehand, then being delivered by railway. Otherwise must be arranged (for large works) a temporary carpenter shop, equipped with circular saw, band saw, planing and boring machines.

Note 2. p. 130. On the larger building sites in some cases quite a number of machines are employed, whose proper arrangement for all later building operations is of the greatest importance. At erection of the Arm Factory in Steyr was produced in one day over 1000 m³ of concrete, which naturally required the arrangement of a very large park of machines. (Z & B. 1914. p. 137).

Sand, gravel and crushed stone, as a rule, are kept apart, heaped in the open air, although wetting by rain makes more difficult the determination of the amount of water required to be added.

Also see the suitable arrangement of a store and work yard for an exchange business of about 1,000,000 marks (storage of abundant steel and wood equipment, as well as of steel machinery and forms for local and foreign structures); *Arm. R.* 1914. p. 107.

Already during the erection of the framework for the forms any test pieces be prepared and tested for strength at the site or at a neighboring laboratory for testing materials. It is then decided, whether the sand and aggregate intended for use, as well as the designed mixing proportions, also correspond to the actual requirements.

b. The Placing of the Reinforcement.

The placing of the steel is preceded by the construction of the forms and framework (see p. 76). The necessary arrangements for transmission are to be provided. Likewise the forms for π chases (pipes, lighting fixtures) are to be properly arranged. Grooves on the surface of the floors must naturally run in the same direction as the bearing rods; see p. 158.

The steel reinforcement is to be arranged ^{so} that its actual distances apart correspond with the calculated distances. This is most simply attained by fixing the position of the steel network on the centering at several points by bits of mortar, stones or small pieces of wood. ¹ In using gravel and crushed stone it is to be considered, that the stones can easily pass between the rods as well as between these and the centering, so that the size of the grains corresponds. During the work of tamping must occur no change in the position of the steel, nor too great deflection, wherefore stirrups and supporting wires are employed, enclosed with them in the concrete. On them use suitable tampers with claw-shaped plates, by means of which these readily pass between the sides of the separate stirrups. The steel rods must not lie too near the surface of the concrete, since then cracks can easily occur in this thin external layer, and consequently the fireproof covering of the steel is destroyed (see p. 5). For columns are first set the vertical rods and then the bands are inserted from above according to the progress of the concreting. Fig. 51 shows an improper arrangement of the bands; the distances must be equal, and the bands must not be oblique. (See Section XV). Great care is to be taken in the placing of the steel reinforcement for thin



slabs. For if the steel in a slab 5 cm thick lies about 1 cm too high, the supporting capacity is reduced about a fourth.

Note 1. p. 131. Very advantageous for girders are small bands of concrete about 2 cm deep, 3 to 4 cm wide and 20 to 30 cm long, placed on the form at distances of 20 to 30 cm. Iron stirrups have the disadvantage, that they show through and produce rusty streaks.

The details of the reinforcement must generally be represented in accurate general steel diagrams. For ordinary floors and piers are first executed the forms either partially or completely, the reinforcement is gradually brought to the desired places, and the concrete is tamped. First are tamped the piers and girders as far as to the underside of the floor, then only are placed the steel rods for the slabs and tamped over. The running planks for bringing the concrete are simply laid on the centering for the floor. If the floor reinforcement (including the distributing rods) be placed at the same time as the reinforcements of the piers and girders, then the running planks must be raised.

One may also first construct the centering and place the entire steel reinforcement at the same time, connected and so wired together, as to produce a steel skeleton as rigid as possible, free from displacement, only then proceeding to the proper work of tamping. In this manner -- previously preparing the steel skeleton on the work yard, and afterwards placing the ready wired parts in the form -- may be attained a cheapening of the labor, an increase in the progress of the building, and a greater certainty. Further see p. 54 and 82.

c. Placing and Tamping the Concrete.

The making of the concrete described in Section III A must occur in the immediate vicinity of the building site, whether it is prepared by hand or machine. A tedious transportation may unfavorably affect the uniformity of mixture by the unavoidable shaking. ⁴⁷³⁸ The carrying of the concrete in hand trays is most advisable, yet in larger works is too expensive. Usually occurs carrying with shovels, indeed just after mixing. For greater areas must be taken wheelbarrows or dump cars, that are taken from the mixer to the place of use. They can hold up to 150 litres, and are provided with pulls in order to be directly hoisted on the building (without transfer to other buckets). In placing the concrete to be used, attention is also paid to

stops at night. Only so much concrete must be prepared as can actually be placed before stopping. Otherwise either the work is to be carried on in the interval, or the remaining concrete is to be excluded from further use. In warm and dry weather, the concrete mass should not remain more than $1/2$ to 1 hour before placing, or in cool and wet weather not over 1 to 2 hours. A longer delay (up to 4 to 6 hours is allowable only with the use of corresponding precautions against drying. All concrete not used at once is to be shoveled over once before its use later (only for subordinate purposes). Bringing with the shovel is not done by throwing, since the uniformity of the mixing would also suffer thereby. Likewise the concrete should not be dropped too far, at most 2 m; for greater heights it is necessary to lower it by spouts or buckets. Boxes with hinged bottoms are employed. -- The less the thickness of the layers, the greater the strength of the concrete. For reinforced concrete buildings is taken 15 (to 20) cm thickness of layers. But if there be no reinforcement, then without detriment one may go to layers 25 to 30 cm in thickness. For example in foundations and walls.

Note 1. p. 133. By experiments it has been actually determined, that by such transportation, in the soft concrete found in the upper part of the wagon -- caused by the shaking -- water rises, the concrete beneath thus containing less water than that above. In such cases a later remixing of the concrete may become necessary. Also see "Transport Concrete," p. 135.

For structures, commercial buildings and warehouses, the cellar is sometimes utilized for setting the mixing machines and the other accessories for mixing. Thus one is independent of all effects of weather, neither suffering from rain, cold or heat. But usually the machine is placed directly beside the building in a shed, protected from wind and weather. The hoisting of the mixed concrete follows by hoists and buckets, ¹ the further transportation in the stories by wooden or iron wheelbarrows; link belt trays are not recommended, since concrete easily falls in the links, then hardens and may disturb the working. For widely extended areas portable mixers are best. One utilizes all advantages of the locality as far as possible, mixes at the higher places, so that the full cars have to travel downward and the empty ones upward, in this way facilitat-

facilitating transportation and thus saving wages.

Note 1. For hoisting the finished concrete by a hoist for materials including driving the stirrer one may count about 20 to 30/100 mark per m^3 of concrete.

In placing concrete in water must first be prevented the washing out of the cement. The trench must therefore be closed against flowing water, and all water pumps must stop during the concreting. The sinking of the concrete sacks has to be done with suitable arrangements (boxes, sacks and directors). Advisable in an addition of trasses, to reduce the washing out of the cement. The concrete itself must only be made earth-damp. (See p. 41).²

Note 2. p. 134. See "Portland cement and its use in construction." 4th edition. 1912. p. 425.

If it concerns the sailing of a particularly great quantity of concrete for a longer period (at least a year), then comes in question the arrangement of a cable railway. One is independent of the country, saves transport laborers, avoids the cost of new roads and the maintenance of streets, shortens the building period, and keeps off the ground and soil of others. The cable railway can at the same time be used for the hoisting and removal of soil, as well as for bringing the centering and framework, the steel skeleton, etc. Adolf Bleichert & Co, Leipzig-Cohlis, supply fixed cable cranes, where a side movement of the cable is not possible, parallel moving cable cranes with side movement of one or two supports, radially movable cable cranes. (Century Hall, Breslau. B & E. 1913. p. 61. Swing-
ing cable cranes with fixed bases, in which the motor rope can be moved sidewise within certain limits.¹

Note 1. p. 135. See Contribs. 1911. p. 133, 140. Ann B. 1914. p. 27, 87.

Magens in Hamburg has discovered a method, that makes it possible to transport ready prepared concrete (the so-called transport concrete) of a definite nature for long distances, then using it with the best results. The distances over which the concrete was taken were quite considerable, in one case being 177 kilometres by rail. The procedure by which Magens treats his concrete is the following:-- a slow setting cement is used, and its preliminary setting is delayed by cooling the materials and by cooling and shaking the mixed concrete. Cooling is done

by cold storage of the materials, by abundant sprinkling with water the working area in the vicinity of the mixers, thus by its vaporisation, further with higher temperatures -- by covering the concrete during transportation with damp sail cloths. The concrete thus made transportable, as several experiments of the Hamburg Technical Experimental Laboratory show, is still usable after transportation for several hours, for ordinary concrete works, and it is even more suitable than concrete made by unskilled contractors at the building site. The concrete is well and strongly shaken together and thereby produces a denser mass in the building. This shaking has aided the tamping to a certain extent. In all cases of transport concrete are advantages for restricted sites, worthy of consideration. Yet before such transport concrete is generally allowed for reinforced concrete structures, thorough experiments are required.²

Note 3. p. 15⁵. Further, see "On Tests of Concrete and Transport Concrete," Deuts. Bauz. Contribs. 1910. p. 61.

Burchartz states on the ground of the experiments mentioned, that by transportation a separation of the mixture occurs, that so ordinary concrete results an average increase of 15 p.c. in strength, and that one either obtains greater safety, or -- especially with aggregate of crushed stone, one can save 15 p. c. of the cement. (B & Z. 1911. p. 245).

R/36. The frequently common method of distribution by gravity for large buildings in America (gravity distribution) consists in this, that the fluid concrete mass (cast concrete, see p. 42) is conducted from a reservoir in an elevated distributing tower through pipes and spouts to the place of use by its own gravity. Tamping does not occur. A few laborers move the movable ends of the spouts to the required place of use. Whether the same quality of concrete is obtained as with us, certainly appears questionable. Advantages of the method; rapidity of execution of the construction, saving in wages, in means of transportation, cars, etc. Avoidance of shocks on completed structural members, saving in framework. Naturally the same equipment may be employed for different structures.¹

Note 1. p. 136. See Contribs. 1911, p. 106. Ann. B. 1913. p. 71. Gent. d. Bauv. 1912. p. 619.

Now commences the tamping of the concrete until it sweats.² If the tamping is to be carried still further, then danger is

incurred (especially with concrete too wet) of producing a softening of the mass under the tamper, thereby disturbing the setting. With concrete too dry, the mass of stones influences the effectiveness of the tools. Quite particularly firm is to be tamped the exterior of the member, since this is most stressed in statical respects. The different tamped surfaces must overlap. Care is also to be taken, that the larger bits of stone do not lie on the exterior of the structure, but are covered by mortar on all sides. Tamping in time is entirely objectionable. Generally the principle is true:-- the stronger the tamping, the greater the strength. ² But the magnitude of the pressure of the tamper depends on whether the tamper is allowed to fall freely and on its weight, or whether the tamper in falling receives an impulse from the hand. Height of fall is about 10 to 30 cm. In all cases the purpose of tamping is to bed the aggregate as closely as possible, and to make the spaces formed between its pieces as small as possible.

Note 2. p.136. According to earlier requirements, the tamping of concrete in earth-damp condition continued until water appeared on the top surface. This regulation is now omitted in consequence of the use of wet concrete. Tamping a soft concrete too long may result in changing the mixture.

Note 2. p.136. By experiment 216 blocks gave about 50 p.c. $\pm$ greater strength than 64 with the same force.

Small vaults are tamped if possible in the direction of the pressure curves, indeed in longitudinal strips, that commence at the abutments and are carried up to the vertex at top before the stop at evening. For flat vaults it is scarcely possible for practical reasons to tamp in the direction of the pressure curves. In any case care is to be taken always, that the tamped layers do not have the same direction as the internal forces, and that the strokes of the tamper do not displace the separate layers. ¹

Note 1. p.137. On tamping in strips for bridge vaults, see Kersten. Arched Bridges, 3rd edition, p. 227.

The tamping itself is done by the so-called tamping tools, that consist of a generally iron plate with rounded edges and a handle fastened thereto. The size of the plate is arranged according to the strength of the concrete for the member treated. Best are iron tampers with square plate of 10 to 15 cm wide

and weighing 12 to 20 kil. Fig. 52). For tamping the external layers of concrete are suitably employed narrow rectangular plates to produce a particularly good tamping. For small and slightly reinforced structural members also light tampers of about 3 to 7 kil weight are to be recommended. For a closer arrangement of the steel reinforcement may be used with advantage heavy round rods with hook-shaped tamping ends; here is rather a tamping of the wet concrete. To level the upper course is frequently employed the concrete plank (also termed tamping plank, etc.) with oblique handle (Fig. 53; the laborer no longer needs to stand directly beside the iron plate. Besides also comes into use the drawn plate. ¹

Note 1. p.138. Also see the Handbuch. Vol. II. p. 157.

For extensive factory use and large building sites, where tamping laborers may be continually busy if possible, recently the use of compressed air tampers finds increasing use. ² While a skilled laborer with an ordinary hand tamper makes about 60 heavy strokes per minute, he can without stress make 400 to 600 strokes in the same time. But with the increase in number of strokes the force of the stroke diminishes. These tampers act in any position, vertical, oblique or horizontal, as well as on parts accessible with difficulty, thus walls and the like. The number of strokes per minute can easily be regulated by a handy air valve. The laborer only needs to move the tool around. He can stop tamping at any time, and at once begin anew. Experiments have shown, that in contrast to hand tamping considerably higher resistance (up to 30 p.c.) can be obtained, that assuming a definite value of resistance, one can save up to 25 p.c. cement. ¹ Furthermore the saving in labor is to be taken into account; for one man here replaces 3 to 4 hand tampers. Fig. 54 shows the use of the compressed air tamper (here without the hose connection). The tamper, as may be seen, consists of a hose supporter for connection of the hose from the compressor, in which the piston with the tamping plate is moved rapidly to and fro, adjusted by a valve. Further statements are found in the "Pocket-Book for Use of Compressed Air", published by the Macalane Shop Co., Frankfurt-a-M., edition of 1914.

Note 2. p.138. The equipment of a compressed air plant consists of:--

a. a compressor, the proper producer of the compressed air.

b. Air reservoir, which maintains equilibrium between the work and the use of the necessary compressed air at the required pressure.

c. The hose leading the compressed air to the place of use.

d. The proper working tools driven by compressed air.

The cost of providing a compressed air equipment (exclusive of motor) for about 3 m³ use per minute must amount to about 8000 marks. (Compressor, reservoir, hose, heads, and 3 tanks). To make its use economical it is naturally advisable to make the utmost use possible of the tolerably dear equipment and working tools. On cost of compressed air equipment, see Z & B. 1910. p. 418, 590. Beton Zeitung. 1912. No. 2.

Note 1. p. 188. Opposed to a disadvantage, that in consequence of heavier tamping, the use on concrete masses is greater than for hand tamping. To this fact must already be taken into consideration in determining the additional cost for concrete work, see Contraba. 1914. p. 84.

Hollow casts and pipes are recently constructed according to the casting process (Dyckerhoff & Widmann Co. Dresden). The procedure consists in using rotating forms, casting in thin fluid cement mortar mixed with asbestos. The casting machine makes 300 to 1500 revolutions per minute, according to dimensions of the pipe.

Note 2. See Z & B. 1912. p. 39 -- Arch. b. 1909. p. 88; 1910. p. 430; 1912. p. 28; 1913. p. 27.

For further construction on hardened concrete (on layers that lie more than 24 hours), the old surface must be roughed with steel brooms, swept clean, wetted, and then coated with thin cement paste directly before applying the new concrete. It is also proper to tamp the new concrete with special force on the roughened surface, so that the union of the two masses may be intimate. Also it is advantageous to take for the first layer a rather wetter concrete or cement mortar (1 cement + 2 to 3 sand), but not neat cement.¹ In all cases it is well to carry on the construction without long stops, so that the number of joints in the layer may be as small as possible. The layers must be made as freshly as possible. Naturally such interruptions are not to be avoided entirely; already the various stops as well as the ending of the day's work make them necessary. It is also appropriate at the beginning of a stop in the work, to cover the ^{exposed} ~~vertical~~ and horizontal surface of the layer

with wet rags, on beginning again to clean and wet it again in a corresponding way, before beginning to lay more concrete. As a rule in buildings the beams are first tamped and then the floor slabs; ² here before all is advisable an abundant arrangement of stirrups and bands in rods to connect together the slab and beam. In construction of piers suffices the extension of the ends of the lower reinforcement (about 40 cm) into the concrete of the upper pier (below). With greater lengths of layers, one can provide for an accurate connection of both surfaces by stepping as in Fig. 55, in a given case with the use of special connecting reinforcement. Yet at the end of the day's work must always be completed an entire tamped layer, so far as no tensile stresses occur there. Otherwise statically perfect separating joints are without importance, when only compressive stresses are found, that act perpendicular to the surface, or when steel reinforcements intersect the separating layers, for example as the case for floor slabs and beams as well as for piers.

Note 1. p.140. It is also advisable to wash the surface of the concrete with dilute muriatic acid, and to remove the old coating of cement, then thoroughly washing off the acid with water, first treating the surface with cement milk.

Note 2. p.140. But whenever possible, beams and slabs should be connected without stopping.

For a natural formation of divisions in the work are recommended first of all the expansion joints. ¹

Note 1. p.141. See Part II. 7th edition. p. 88.

d. Precautions against Frost and Heat.

Although Portland cement of all hydraulic cements is least sensitive to the influence of frost, concreting in winter must still be done with care. A long continued period of freezing (more so repetitions of short periods of freezing) can only be injurious to concrete, when the cement mortar has not yet set and is too damp on top. That is changed into ice, for the freezing of the necessary hydrating water may produce a disintegration of the mass, ² but in every case lengthens the times of setting and hardening; for cold retards and heat hastens the setting and hardening of concrete. The resistance to crushing under the influence of frost is at first somewhat less than for protected concrete, but already in about 90 days it attains

attains pretty near the strength of the protected concrete. The resistance can only be greatly reduced, if the frost acts on ~~an~~ entirely fresh concrete. If this concerns thicker and more massive structural members, even greater freezing forms no direct danger. Before all one should lengthen the time before removing forms, so that the heat arising from hardening may be retained. The main thing is that the concrete has time for later hardening in ordinary warm weather. Freezing has no injurious effect on already hardened concrete.

Note 2. p. 144. Usually the consequences of a sudden frost do not show much in the appearance; for by the freezing in the not yet combined water will the process of setting be stopped. A temperature of 0°C lengthens the setting period of the cement to 3 or 4 times the ordinary setting period. If it thaws ~~again~~, the concrete sets further. Only alternating freezing ~~and~~ and thawing is not good. -- Experiments of Burchartz showed ~~that~~ that several hours of freezing did not essentially affect the hardening of mortar and concrete, but that long continued frost materially delayed the progress of hardening, and indeed so ~~much~~ much the more, the longer the effect of frost lasted. With long continued frost, especially for a lean mixture, the favorable effect of a greater addition of water made itself felt more strongly than by a short duration of freezing.

p. 144 Concrete work with 5° to 10° C of frost is to be forbidden, if no special precautions are taken. Yet if one must continue work in severe cold, -- to keep within the contract time for completion, -- then (particularly in bridge work) one can erect construction sheds, which are to be warmed by stoves day ~~and~~ and night. Under some conditions a covering of the structure with sail-cloth would suffice. Then the concrete is not subjected to stress too soon. Naturally plastering must never be done in freezing weather.

One may also aid, when the placing of the concrete is undertaken in dry interiors protected from the wind, by heating the water, sand and aggregate before using (up to 40° C),¹ then ~~limiting~~ limiting the addition of water to a minimum, and finally protecting the completed and tamped concrete by bad conductors of heat, like straw, hay, sail-cloth, sawdust, sacks of sand, stable manure,² rotten leaves, etc., over this being laid sail-cloth, empty cement sacks or tarred roofing felt with timbers

laid thereon -- from the direct effect of frost. But in the frost period is not allowable both the use of frozen stones mixed with bits of ice and snow, as well as the pouring on of the cement mortar for shortening the time of hardening, or an artificial heating for the same purpose. All the precautions mentioned are then especially of great importance, if the structural members to be protected are of small dimensions, and are statically stressed high; economically they are not as injurious as long interruptions of the work.

Note 1. p. 142. For a lesser degree of frost it is generally sufficient to warm the water only. Cement is heated in no case.

Note 2. p. 142. Manure is especially good in consequence of its own heat, yet direct contact with the concrete must be avoided.

Sometimes, so far as it concerns purely lamped concrete structures, ³ to the water for the mortar is added some common salt (up to 20 p.c. solution) to accelerate the setting, but which under some conditions may cause efflorescence. ¹ Better is a smaller addition of soda or also of lime chloride. ² (50 p. c. of the water added). Works during frost naturally -- as far from loss of time -- result in many increases in cost of the procedure; removal of snow, thawing the ice covering (with salt). Breaking up and thawing the frozen lumps of gravel, etc., all works that are to be previously accounted for in the bid, as far as possible. It is even recommended to mix somewhat richer in freezing weather, since then the effect of frost is not so unfavorable. With not sufficiently rich mixture in a given case is advisable a previous coating of the steel with cement milk (p. 50) ³ -- Also no test cubes are made in frost, so far as by the building superintendent no corresponding increase of the hardening period is allowed.

Note 1. p. 143. The use of salt is just as simple as cheap. About a handful is taken for 57 litres of water. The water may be warmed previously, so that the salt dissolves more quickly. Yet two evils exist. Sometimes the above mentioned efflorescence appears, a whitish exudation (wrongly termed "masonry saltpetre"). This exudation may be removed by washing with a dilute solution of muriatic acid and afterwards abundantly with clear water. Finally the ^{on account} of the salt contained, the concrete never becomes entirely dry, since all salts absorb water to a great extent. Further see B & E. 1911. p. 83.

Note 2. p.143. As "Kalzium" in the trade. But an increasing proportion of it reduces the tensile strength of the concrete, less so the compressive resistance. Also the preparation of the concrete is made more difficult.

Note 3. p.143. According to the experiments of Gerger (Contribs. 1911, p. 155, 191), coating the steel with cement milk increases the bond resistance, thus the bearing capacity of the beams to a substantial degree in the effect of frost. Merely setting the steel with water almost entirely destroys the bond resistance during a frost of eight days.

At the erection of the Mountain Hotel in Sommerberg in the Black Forest, concreting was even carried on at -16°C . (Z & B 1909. p. 275).

In Z & B, 1909, p. 51, is mentioned a case in which frost at -5°C was overcome by steam, so that it produced no delay at all. Scantlings were laid on the finished concrete floor and covered with boards, so that over the concrete was a connected hollow space. The boards were then covered with doubled sacks, and for 5 or 6 hours steam was introduced into the space by temporary piping from a steam engine. The floor absorbed so much heat for its entire thickness, that after removal of the covering the hardening proceeded with extraordinary rapidity.

The injurious effects of frost may also be otherwise recognized, in that the concrete often sticks to the centering and the surface shows cracks.

p. 144
But not only the cold, but too great heat may injuriously affect stamped concrete. Warm weather accelerates the hardening and leads to a greater final strength; only the setting must occur in moderate weather. A high degree of heat has first the evil as a result, that the top layers set considerably quicker than the lower ones. Therefore sufficient care is to be taken for a frequent sprinkling with water as well as a protection from the direct effect of the heat. The concrete is to be made with more water, also instead of the increased sprinkling, one can lessen the unfavorable effect of heat by covering with wet sand. Thereby a too early evaporation of the pure water in the cement is prevented. Otherwise cement sacks or boards are laid over it, which can retain the dampness for a considerable time. One also takes account by structural arrangements of the change in volume of the combined members in consequence of the change in temperature, for example, by forming extension joints

(expansion joints), produced by the insertion of wood, sheet or metal, roofing felt, etc., and which serve to prevent expansion cracks. ¹

Note 1. p.144. On the construction and arrangement of such expansion joints, see Bersten, Reinf. Concr. Part II. p. 36.

According to all this, great heat by sunshine, as well as great cold are injurious to the setting. The best seasons are spring and early autumn, the worst being late autumn and winter. ²

Note 2. p.144. Further see Reft 12 of Publications of D. A. 1911. (Experiments on the effect of cold and heat on the hardening capacity of concrete); Betonbau. 1912., p. 32; Clay Industries-Zeitung. 1912. No. 115.

e. Removal of Centering and Forms.

After completion of the tamping all shocks and vibrations are to be kept away from the building. Slight vibrations during setting are without consequences. The concrete must not be disturbed and not be loaded with building materials or the scaffolds of the upper stories before it is hardened. A frequent sprinkling with water in summer (for 8 to 10 days) accelerates the hardening. Likewise the direct effect of heavy winds must be prevented. Further it is to be seen carefully, that no holes and chases for the reception of cables and pipes be not formed in places, that must suffer no weakening on account of important internal stresses. ¹ Also one does not use the fresh angle of the concrete in removing forms as a fulcrum for a crow bar. To avoid vibrations, instead of hails is recommended the most extensive use of screws, bolts, clamps and the like.

Note 1. p.145. Also see p. 158.

The correct time for removal of centering and forms depends entirely on the goodness of the concrete, on the later purpose of the structure, and not for the least part on the prevailing weather conditions. In autumn, days and nights are colder; the concrete sets more slowly, and the period of hardening does not pass so rapidly as in summer. Likewise the materials at delivery are colder than in summer, and also the water; cold hours of the night chill the concrete. All delays the setting of the concrete, often in considerable measure, wherefore the centering of the floors and vaults must remain longer in cold weather to prevent accidents. ² Gravel produces a concrete somewhat slower in hardening than crushed stone. It is absolutely necessary, that the

necessary, that the removal of centering should always be left to a skilled specialist or foreman. For most accidents, that have occurred in reinforced concrete construction, are to be referred to a too early removal, or to an improper and unsystematic removal. (See p. 158).

Note 1. p. 145. It is not always advisable to make the date of removal depend on the rate of hardening of a test block of like age (p. 127); for the conditions of hardening are of a different nature for both cases. (Sectional dimensions, effects of wind, weather, etc.)

An early removal of the centering presents the advantage of a quicker use again of the planks and timbers of the centering, but often results in the great disadvantage, that in consequence of insufficient hardening of the concrete, changes in the forms of the structural members may occur. If necessary, by striking with a hammer or by scratching with a sharp tool, one may thereby become convinced, whether the removal can be begun.

The Prussian Regulations prescribe:--

"The removal of side forms of beams, and of the forms for supports, as well as the removal of the centering of the floor slabs, must not occur before the lapse of eight days, nor that of the supporting forms of beams before the passage of three weeks. ¹ For greater spans and dimensions of sections, the time is to be increased to six months under some conditions.

If the tamping ends but a short time before the occurrence of frost, then special providence is to be exercised in the removal of the centering and of the supports. If a frost occurs during the time of hardening, then with regard to the fact, that the hardening of the concrete is delayed by frost, the time is to be extended by the duration of the frost.

For buildings of several stories, the supports of the lower ceilings and beams are only to be removed, if the hardening of the upper ones has advanced so far, that they are able to support themselves. ¹ In the construction of walls and piers in buildings of several stories, the construction must be commenced in the upper story only after sufficient hardening of these architectural members in the stories lying beneath it. Of the beginning of the work in the higher story, at least three days' notice is to be given previously to the building officials."

Note 1. p. 148. Austrian Regulations require throughout 4 weeks time before the removal (removal of supporting timbers).

Excess regulations make the time dependant on the dead weight concerned, and specify:--

10 days for spans up to 3 m.

20 days for spans up to 6 m.

30 days for spans over 6 m.

Non-supporting centering timbers (wide forms of beams) may be removed earlier (often about 3 to 5 days). The floor of an upper story, as a principle, should only be tamped when beneath it exist two complete floor centerings, unless the new floor centering rests on the ground or on a similarly firm base (masonry); for a floor under construction transmits not only the weight, but also the shocks connected with the tamping. -- Not all the supports of a girder are to be removed at once, but gradually, such if the girder be fully hardened already. For buildings in several stories, the removal of the supports are made from above downward, as a rule. See Note on p. 147.

Note 1. p.147. Every floor from which the centering is removed must be in condition to receive the load of the recently constructed floor lying above it with sufficient safety. It is advised to begin first with the removal of the centering of the lower floor only when the hardening of the upper floor has progressed so far, that it can support itself. See Note on p. 146.

The sequence of work in removing the centering of an ordinary floor is:-- assuming favorable conditions of weather -- generally the following:.

In 5 to 6 days:-- removal of forms of supports; ² loosening wedges of the floor struts and removal of these struts; removal of top and side forms of cross and main beams; ³ renewed placing of some necessary struts under middle of floor as in Fig. 56 (so far as the floors of a building in several stories must be loaded early. ⁴

Note 2.p.147. The angles of fresh concrete, at least in the lower parts, must be protected against injuries of all kinds by special covering boards.

Note 3.p.147. The side forms of beams and supports can frequently be removed earlier, assuming sufficient hardening of the concrete.

Note 4. p. 147. If the just finished floors must be used for building materials, then are employed properly connected walking planks, to avoid vibrations.

In damp weather, in cold or with partially prevented access of air, under some conditions twice as much or more time is necessary. All removal of centering occurs in panels, beginning at the wall. Then more with the lever than with striking tools (hammer, sledge) the wedges under the struts (Fig. 35) are to be loosened very carefully. The time for removal must be larger, the greater the distances between supports, and the dead weight of the floor, as well as the stresses in the structural materials. Great frame and vaulted structures, bridges and the like cannot be cleared of forms frequently before 5 or 6 weeks.

f. Superintendence and Acceptance of the Building. (Test Loads on completed Portions).

Some of the most important problems for the local superintendent are the following:--

Suitable arrangement of the building site.

Oversight of the acceptance and storage of building materials, especially of cement, storage of steel by kinds and lengths, and checking by calipers.

Testing steel by bending (if necessary left to supply dealer).

Suitable distribution of work according to plan of concreting.

Selection of suitable foremen for the different groups of laborers.

Supervision of centering and forms, mixing and placing concrete, placing of steel reinforcement, removal of centering, etc.

Keeping a journal with statement of daily work undertaken, time of construction, removal of forms, daily temperature, days of frost and rain, etc.

The local superintendent must be in permanent relations with the supervising engineer (telephone connection between the building site and the business office of contractor is quite proper for larger buildings; he must always be at the place; a change in superintendent is to be avoided. The supervising engineer has to frequently visit the building and to go through all building plans with the local superintendent (p.158). Naturally the local superintendent must be instructed concerning all exchange of letters and conversations of the supervising engineer with the owners of the building.

Reliable archives are formed, that among other things contain exact statements on the assumptions in calculations, mixing proportions, and on the concealed steel reinforcements, so that later for alterations and extensions, even after a long time

may be found reliable data.

p. 149. If a well based assumption exists, that defects are contained in the completed structure, which reduce the required bearing capacity in consequence of later influences, the building officials may decide a local testing to be required. If design and construction under intelligent oversight entirely correspond to the official regulations, the detailed and costly ¹ test loadings are properly necessary only in bridge construction and not in buildings, in any case only when slightly tested architectural forms are concerned. Test loadings and measurements of deflections are and remain tolerably worthless as a standard of the quality of the concrete construction. ² Far more important is a conscientious superintendence and a frequent testing of cubes and beams. (p. 119).

Note 1. p. 149. The cost of the loading test, that generally the contractor must bear, must be in a suitable proportion to the total cost of construction of the building, at most reaching about 1 p.c. of the contract price.

Note 2. p. 149. It should not be omitted, that such loading test easily results in overstressing the building materials, and may occasion hair cracks. (The requirements of the Railway Direction at Berlin speak of "actual" cracks in contrast to $\pm$ the so-called hardening or air cracks). -- One should never attribute too great importance to the first fine cracks; so far as they do not become larger (by adding or removing loads), they are without importance for the resistance of the concrete. Naturally they should not widen, so that the reinforcement is bored. Reinforced concrete structures without fine cracks in the surface are generally but seldom found. Experiments have shown, that the number of cracks may increase, if the same cross section of steel is composed of a greater number of smaller rods, even if the steel has rough surfaces, since then the concrete cooperates more in working together. B & E. 1909. p. 175, 263, 275, 302; Arm. B. 1909, p. 232, 30, 60.

If loading tests are not to be omitted, then the requirement of 45 days' hardening is to be taken as the least requirement. Decidedly better is 90 days' hardening, as the French Regulations provide. Otherwise one is dependent in this point on the progress of the building.

On load tests of entire floor panels and beams, the following is prescribed in the Prussian Regulations:--

"In loading floor slabs and beams, if g = dead load and p = uniformly distributed live load, the loading shall not exceed $0.5 g + 1.5 p$. With live loads greater than 1000 kil/m^2 may be made reductions to the simple live load."

According to the "Preliminary Principles," which in many respects were taken as a model in the revision of the new Prussian Regulations, the loading must not exceed $0.5 g + 1.5 p$.¹ (With a slab 10 cm thick and a live load of 500 kil/m^2 , according to the official requirements, the loading $p = 0.5 \times 0.10 + 2400$ ~~mm~~ $+ 1.5 \times 500 = 870 \text{ kil/m}^2$, and according to the Preliminary Principles $p = 0.5 \times 0.10 \times 2400 + 1.5 \times 500 = 1092 \text{ kil/m}^2$). Very proper is the reduction for live loads greater than 1000 kil/m^2 , since otherwise for such a loading unusual loading masses would be required, and the loading test might become dangerous. One must first of all avoid exceeding the elastic limit of the steel by the stress in it (about 1800 kil/cm^2). For economizing cost the ceiling of the cellar will be chosen for the loading test of a building of several stories, since there the mass of the loading can be more easily and rapidly placed.

Note, p. 150. For such supporting structures, that are so connected with each other, that a partial participation of the latter in supporting the load may be expected, these shall be:-- live loading = $1.5 p$. live load = $1.5 p$ for independent supporting work not connected with others.

Austrian Regulations. live load = $1.5 p$. (likewise Rurtemberg, Saxony and Danish Regulations).

French Regulations. live load = $1.0 p$. (Very mild rule, likewise Hungarian Regulations).

Duchy of Brunswick. live load = $0.25 g + 1.25 p$.

In the determination of the increased live loading and carrying out the entire loading test, it must also be considered, that the stress on the foundations must not exceed the limits, which are assumed statically permissible. In artificial foundations, particularly pile foundations not reaching the solid ground, the justification for undertaking a loading test may generally appear questionable.

On test loadings on a separate strip of floor,² the following is prescribed.

"If only a strip of floor panel be loaded for a test, then is the live load to be uniformly distributed on a strip in the

middle of the floor, whose length equals the span width, and whose width is one-third the span, but at least 1 m. The loading shall not exceed $g + 2 p$. As dead load is taken all the structural parts intended for the construction of the ceiling and floor, as live load the increased values in Section XVI, 3.

The usual loading tests are based on the fact, that any fracture is preceded by an elastic change of form, by a deflection for floors and beams. If now this elastic change be accurately determined by the loading test, then may one deduce a conclusion in regard to the resistance of the part of the building concerned. ¹ The loading test itself is made by piling steel bars, bricks, sacks of cement or sand (each weighting 50 to 70 kil), if possible on a layer of sand 5 to 10 cm thick to distribute the load uniformly. One may also use steel tanks, gradually filled with water. ² Bricks must not be loaded; instead of a uniform loading, the live loading may arch over the floor, and the greater part of the load act directly on the supports. Better than a uniform distribution of the loading is the arrangement of the two concentrated loads near the middle of the floor. Simplest is naturally a single load at the middle of the beam; only half the loading is required. For the use of rolled beams, it is preferable to place steel rollers between the end bearings of the beam to avoid any fixing.

Note 1. p.151. Yet the amount of deflection depends in a certain degree on the arrangement of the reinforcement (whether with or without stirrups, with or without bands, etc.). These loadings thus present no correct data for sufficient prediction with regard to shear and bond resistance. -- Further it is to be considered, that in consequence of the connection with the structural parts, subordinate stresses exist, that have a much influence the results.

Note 2. p.151. Clay Industries Zeit. 1912. No. 52. -- On making loading tests, also see B & S. 1912, p.288; 1913, p.95, 96, 142, 416.

The separate loads must occur at certain intervals. The magnitude of the loads as well as the changes of form read off are to be accurately recorded, and likewise are sketched the hair cracks corresponding to the development of deflections. The last reading of the apparatus ¹ should only be made some time after the removal of the load. The measuring apparatus

of observation and is so connected with the stretched wire, ϵ that the latter lies between two rollers of aluminium bronze as hard as steel and is tangent to them. (Fig. 57). Griot's Δ deflection measurer is also adapted for sidewise bending of supports. (Fig. 58).

Among other flexure measurers are Martens' mirror apparatus, the flexure indicator of Osske, Bauschinger's measurer, Fränkel's indicator, etc.

If one has a floor construction like Fig. 59 to examine under the required loading, then must be measured:-- ⁵⁹ 1

- At a the deflection of the floor.
- At b the deflection of the smaller beams.
- At c the deflection of the main girder.
- At d the flexure or bending of the supports.

Note 1. p.153. But it is to be considered, that the deflection determined at b already contains a part of that produced at c. Finally, it is not to be forgotten, that a compression of the soil may occur under d.

For specially constructed test pieces (slab of floor, etc.), the loading is to be carried to breaking; then from the ratio of the breaking load to the bearing capacity of the floor may conclusions be deduced with reference to sufficient safety. Likewise here also are the deflections at the different sections of the test piece are to be measured and drawn. ¹ But as a standard for the quality and bearing capacity cannot be taken the occurrence of the first crack; for it alone is determinative of safety against breaking. ²

Note 1. p.154. Also see "Loading test of untested structures" Eisenbeton. 1909. p. 229; Contr'bs. 1912. p. 75. Building officials can here make the permit dependent on the results of previous tests and loading experiments. The loading experiments are to be carried to breaking.

For loading tests to fracture, the sum of the permanent ^{dead} ~~static~~ load and the applied loading must at least equal threefold the sum of the dead load and the prescribed live load. (Swiss Regulations).

The load producing fracture must at least correspond to three times the live load + twice the dead load of the supporting member + thrice the other permanent loading. (Austrian Regs.).

Note 2. p.154. The Wurtemberg Regulations of Dec. 1912. pres-

b. Load concentrated at middle: $\delta = \frac{\sigma_c l^2}{2520 (h' - x)}$.

c. cantilever uniformly loaded: $\delta = \frac{\sigma_c l^2}{840 (h' - x)}$.

Note 1. p.155. As an example (Fig. 63), if $x = 11$ cm;

$$I_b = \frac{200 \times 11^3}{3} - \frac{180 \times 1.0^3}{3} = 88,673 \text{ cm}^4$$

$$I_c = 28.3 \times 26^2 = 18,960 \text{ cm}^4$$

$$I = 88,673 + 15 \times 18,960 = 373,073 \text{ cm}^4.$$

Note 2. p. 155. The formula is based on the free support of the ends of the beam. If fixed at both ends, multiply by 1/5; if fixed at one end only, multiply by 3/5. For deducing the formulas, see B & E. 1908. p. 225.

For the deflection curve is here assumed the parabolic form of the elastic curve. l and $(h' - x)$ are expressed in cm. But the computed deflections are always greater than the actual ones, since here also the tensile resistance of the concrete is not considered. ¹ Yet if a comparison is not found in favor of the observed deflections, then is to be presumed a defective execution.

Note 1. p. 156. The formulas can also be employed, if one wishes to take into account the tensile resistance of the concrete. Then are changed the values $(h' - x)$ and the stresses, and thereby also the deflection d .

Further see B & E. 1909, p. 291; 1910, p. 257; 1908, p. 398; 1909, p. 22; 1914, p. 91, 182, 225; 1912, p. 20. Eisenbeton 1909, p. 130, 141. Bunds. f. Tech. u. Wirt. 11, No. 5. Schweizbauz. 1914, Vol. 63, No. 40.

SECTION V. ACCIDENTS TO BUILDINGS AND REBUILDING.

At the end of 1911 were introduced statistics of accidents by the German Commission on Reinforced Concrete, and in Prussia the Minister of Public Works gave to the building officials instructions by which thenceforth all accidents to structures of reinforced concrete must be investigated by competent specialists.

A. Causes of the Fall of Buildings.

Note 2. p. 156. Also see the lecture by Professor S. Müller. "Building Officials and Failures." B & E. 1912. Appendix. Also see Contribs. 1912. p. 51. Arm. E. 1912. p. 314.

1. Statical Defects in Design. (Great numerical errors are seldom original causes).

Defective consideration of the shears, that by the occurrence of any defect in construction mostly have an effect more unfavorable than the bending stresses.

Assumption of actually non-existent fixing at the supports.

Erroneous assumption of loads (for example, thicker slabs than assumed at first). Incorrect estimation of the movable ^{P. 157} live load. Later loading by machines, not included in the statical loading of the floor, or only later indicated wooden superstructure on the roof trusses.

Later recalculations and changes in the design, and during the construction by the architect or owner, which often have a great influence on the statical calculation. Substantial changes in positions of beams, that are frequently not noticed in the local checking. Later reduction of depths of beams, ¹ mostly at the desire of the architect. (Too numerous rods with too little distance between them, Fig. 64).

Note 1. p. 157. Frequently in the description are given irrational requirements concerning the depth of beams. This depth must always be in a certain proportion to the span.

2. Structural Faults in Design.

The heavier structural members are not only to be checked by calculation, but also by a specialist.

3. Faults in the Construction of the Building.

Use of defective building materials (loamy, untested gravel sand, etc.). Defective care in mixing and tamping, ² displacement of the reinforcement in concreting as in Fig. 65 (large strips of floors without reinforcement, etc.). Unpermitted sa-

esteem themselves sufficiently competent to superintend as skilled foremen the construction of reinforced concrete buildings; the supervision is just as insufficient as the lack of building officials, and evil results cannot fail. An inexperienced contractor with good assistants can produce better work than a specially trained contractor with inexperienced laborers and foremen. The Swiss Regulations therefore justly require, that only "permissible foremen must be employed, who possess experience in this system of construction."

According to the Dresden agreement for wages, of Aug. 9, 1910, a distinction is made between special workmen in concrete construction:--

a. Entirely skilled cement workers, i.e., workmen, who can prove at least four years of work with cement and nettings, and are able to work correctly from drawings, to select the steel, bend in place and wire it, to work and treat the concrete correctly and properly, to lay off a ceiling plane and straight and to finish it, to construct a floor at pleasure with any subdivision by joints, to further plaster and smooth, and in general can work independently.

b. Cementer and netting worker, i.e., workmen, that can execute only a portion of the previously named work, and that can prove the practice of this activity for at least one year.

Note 1. p. 159. "The contractor has to place for the direct supervision of the reinforced concrete work an engineer experienced in this method of construction, during the construction. At the demand of the building officials, the contractor is required to prove, that the person entrusted with the conduct and superintendence in the erection of reinforced concrete work has already been employed successfully." (Württemberg Regulations. Dec. 1912). -- If a specialist is placed in local charge of the construction, then the higher supervision must contain the same engineer, who had charge in the office of the calculations and drawings, who conducted the business with the officials, and who also made out the bid and the estimate of cost. Also see the Essay, "Duty of License for Concrete Work." B & E. 1911. p. 76.; 1912. p. 299.

Extremely limited building period (pressure of owners for completion).

Reduction of cost in competition (saving in the wrong place).

disadvantage of public competitions, in which unacceptable contractors can participate, frequently receiving the award by the cheapest bid.

None or too late checking of the statical calculations after letting the contract.

Lack of technically skilled men in the building office, who have to check the building project not only by calculations, but also structurally, and who know the practice of reinforced concrete construction by their own experience. ²

Note 2. p. 159. Professor S. Miller recommends the rearrangement of statical offices. These requirements have already been fulfilled in various places. In all cases the officials of the technical societies -- which was already considered -- were unable to prevent bad constructions; these officials mostly lack the special knowledge. (B & E. 1913. p.28, 200).

5. Influence of Higher Powers.

Unavoidable accidents in which forces come in question, which are beyond calculation, for example earthquakes (p. 14), explosions (p. 14), floods, broken dykes, fire (p.3), lightning strokes (p. 109), etca, as well as accidents during the work, unforeseen falling loads, movements of foundations by the sinking of the ground water, appearance of springs, vibrations and the like.

p/60
By the German Concrete Union was introduced in 1910 an arbitration decision, that besides the arrangement of fees also contains a list of competent persons,. By means of this list suitable persons can be found at once in the vicinity of the accident, who are capable of clearly establishing the cause of the accident, and when engaged, these are to make this public in the shortest time. The appointment of such a board of arbitrators is the more in place, that the abundance of technical questions in court proceedings as well as the hearing of experts is made necessary, where it is to be considered, that by the length of time over which the hearing of experts extends, ^{proofs are made} that the ~~establishment~~ quite difficult. It is beyond all doubt, that the establishment of this arrangement of a board of arbitration by the German Concrete Union would have an extremely beneficial effect, and numerous court fees and much precious time would be saved.

B. Formation of Cracks.

The before mentioned causes of accidents first allow the formation of cracks in the ^{structural} ~~members~~ members in sympathy. Naturally the live cracks (cracks that extend farther) are more dangerous than the dead cracks, that have come to rest. ¹

Note 1.p.160. Every crack is nowise dangerous. In this respect one should not be excessively anxious, and should consider, that under the same causes other structural materials, like ^{comparison} wood, steel and masonry in ~~compression~~ with reinforced concrete with its smooth upper surface, all appearance of injury and cracks are recognized with much greater difficulty or not at all.

Injuries to supports by cracks are extremely rare.

Injury by cracks in the slabs of ribbed floors; mostly entirely harmless shrinkage cracks (a) parallel to the reinforcement or slight angle cracks (b). Only separating cracks (c) may be dangerous and make strengthening necessary.

Injury by cracks in the ribs of ribbed floors. Vertical cracks at about the middle of the beam are mostly unimportant tension cracks in the concrete. Horizontal cracks correspond to the joints in the work and only make strengthening necessary, when they extend deeply. Entirely dangerous are only the oblique ^{p.161} shear cracks in the vicinity of the point of zero moment, that so far as they extend to the underside of the beam, are also visible on the bottom of the beam.

C. Rebuilding Works.

In consequence of the strength of the reinforced concrete the number of such works is extremely small. They frequently require a preliminary relief from the loading by wooden scaffolds; they likewise generally need extensive shoring. For the following are given some examples of reconstruction work. ¹

Note 1.p.161. See the Lecture of Professor S. Müller, "Reconstruction Works in Reinforced Concrete" in the Annual Assembly of the German Concrete Union in 1914; Arm. B. 1914, p. 138, $\pm$ 178, 208.

Fig. 67 a; around the old pier is tamped a new concrete ring.

Fig. 67 b; reduction of distance between supports in the lower story by insertion of a new support (in stone, steel or reinforced concrete). Disadvantages; new negative moments without corresponding steel reinforcement.

Fig. 67 c; insertion of steel beams below and beside the old.

Disadvantages; loss of height and width; no correct statical effect; difficult extension of the beam to the wall bearing. Best are two side plate beams as in Fig. 67 a with cross connections through the concrete beam (less loss of height and width).

Fig. 67 e; bridging over these cracks by new rods covered with concrete (bends placed close together). Good statical combination, especially when the side strengthening rods are connected together through the beam.

Fig. 67 f; box enclosure according to Professor S. Müller. The old beam -- after thorough wetting -- is enclosed by a new concrete covering with continuous oblique enclosure of round steel rods. Two holes extend beside ~~each other~~ ^{the rod and} serve to carry the steel up to the upper zone of the beam. The concrete is then applied in thin fluid form. Advantage of this covering; ease of use, cheapness, executed in any lengths, rapidity (a beam in a day's work).

Fig. 67 g; diagonal lattice in the interior of the beam according to Professor S. Müller, (for broad and heavy beams). In stead of the U-form one strengthening rib has ~~an~~ ^a T-shape. The web lies in the middle of the beam, the strengthening rods pass again through holes, that are conically enlarged downward; the bottom of strengthening flange is thus toothed into the upper beam.

Fig. 67 n; increased thickness of slab, roughing and soaking the upper surface; then a new tamped layer at least 4 cm thick, with steel network (stirrup connections with the tension rods of the old slab are preferable. ¹

Note 1.p.182. On similar strengthening works, also see B & E. 1914, p. 97.

Fig. 67 i; new connection of a separated floor slab at the compression zone of the beam (cracks c in Fig. 66). Cantilever rods in longitudinal grooves connected with the lower reinforcement.

II. Structural Development of the Basal Forms, Calculation of External Forces and Bending Moments. Spans of Floor Slabs.

Between permanent loading, i.e. dead weight (beams, suspended Rabinz ceiling, etc.) and live loading, i.e. the traffic loading.² With the existence of concentrated loads, their most unfavorable position is introduced in the calculations. Some snock is considered by corresponding additions to the existing uniformly distributed loads are to be assumed according to the Prussian Regulations.

By live load may also be understood the traffic load (floor etc.).

For members subject to moderate vibrations, i.e. floors of houses, business offices, warehouses, etc. existing dead and live loads.¹

Very worthy of consideration is the following regulation of the English Regulations:-- for columns or piers in relation with three or more strong floors, the loading in the different stories is to be obtained as follows; for the uppermost floor is to be taken the full accidental load in addition to the live load assumed for it; for the floor next below the uppermost is to be taken 10 p.c. less than that assumed for the uppermost; for the next 20 p.c. less, and so forth down to the story, where the reduction amounts to 50 p.c. of the assumed load of the story. For all lower stories the accidental load of the columns is taken at 50 p.c. of the load assumed for the story."

B. For members exposed to stronger snocks or vibrations, i.e. strongly varied loading, as for example, floors in assembly halls, dancing halls, factories, storehouses; the actual dead load and the live load increased to 50 p.c.²

Note 2. In factories is to be taken into account the exact arrangement of heavy machines. A later transposition of the machines may be dangerous, and at least may afford opportunity for the formation of too large cracks. -- Otherwise the vibrations only come into consideration for the slabs, less so for the beams, and scarcely at all for the supports.

C. For members with strong snocks, for example for cellar ~~ceilings~~ and ~~under~~ ^{ceilings} and courts; the actual dead weight and the live load increased to 100 p.c.³

1904, p. 105. In a given case the rolling concentrated loads (weight wheels of heavy wagons) must be especially considered. It is to be remembered, that 100 p.c. addition for strongly stressed structures is itself taken very high; nearly all regulations have been satisfied with at most 50 p.c. Some require no addition to the live load, but a reduction of the stresses in the concrete and steel. (See Kärntner Regulations, VII.4).

"The live load, besides the weight of the supporting structural members, as it be taken the weight of the building materials serving to form the flooring by well known unit data."

More accurate data on the values of floor loads are to be taken from a table contained in the Appendix. ⁴ The dead weight of the concrete is to be assumed at 2400 kil/m^3 according to official regulations, including the steel reinforcement, when no other weight is directed. Yet in many cases, especially when the steel reinforcement is strongly developed, that it must support with the centering, it is advisable to obtain the dead weight by experiment. The weight of a unit volume of the concrete itself varies between 2000 and 2300 kil/m^3 (see p. 75). Lighter and worse concrete (good as filling) with a weight of not over 2000 kil/m^3 , as already stated, must find use in reinforced concrete construction only after preliminary tests. (See p. 31, 32).

Note 4, p. 105. By the Prussian minister of Public Works was issued a decree on Jan. 31, 1910, in reference to the determination of live loadings to be assumed for buildings. (See the Appendix of this volume). According to this decree, for wind pressure normal to a plane surface inclined α° to the direction of the wind is to be taken, $W = p \sin^2 \alpha^\circ$. (Lössel's formula); formerly Lössel's formula was valid, $W = p \sin \alpha$. If α equals or is less than 25° , the horizontal component is to be neglected.

Thus load on the inclined roof, $S = 75 \cos \alpha^\circ \text{ kil/m}^2$ of ground area. According to Schaller (Loading of architectural structures by snow, K. Ernst & Son, Berlin, should be used:--
 70 to 75 kil/m^2 for regions up to 200 m above sea level.
 120 kil/m^2 for regions 200 to 500 m above sea level.
 340 kil/m^2 for regions 500 to 2000 m above sea level.

According to the Swiss Regulations the dead weight of the concrete is to be taken at 2500 kil/m^3 , indeed with regard to the inevitable and unavoidable increase in thickness in conse-

as small as possible.

For continuous beams according to all regulations $l = L + b$ is to be assumed. Yet particularly for small values of L but great dimensions b of piers, it must suffice to assume $l = L + b/2$ for the length in calculations. (Fig. 69).

b. Slabs freely supported and those with fixed loading.

The rods, that have to receive tensile and compressile stresses are called the bearing rods of the structure. If a slab rests free at all sides, these rods are properly placed in the direction of the smaller span. For an approximately square plan is advisable a crossed position of the rods. ² In simple buildings round rods of 8 to 15 mm are usually taken and placed at distances of 8 to 15 cm; distances below 8 cm are not recommended on account of difficulty in tamping. Bearing rods should never be spliced at points, where great bending moments occur. For great spans and loads the thickness of the slab is to be made quite great, or the steel reinforcement is to be made very strong. Otherwise one always does well to employ rather small distances between rods and small rod sections, than great distances and large sections. At the ends the rods are to be bent as in Fig. 70. (Also see p.52.- Further see Section XII.).

Note 2. p.165. See Section VI c.

In contrast to the bearing rods are the so-called distributing rods. These connect the former transversely and have the purpose of holding the bearing rods more securely in tamping, of distributing among those uniformly the effect of the external forces, of preventing shrinkage cracks in the direction of the steel reinforcement, and of increasing the resistance to shear of the concrete mass. On account of their smaller importance smaller sections are given to the distributing rods than the bearing rods (6 to 10 mm), and they are laid over those, so that the bearing rods may lie as near as possible to the outermost, thus the most strongly stressed layer of fibres. According to the official regulations, there must exist under the bearing rods in the beam at least 2 cm thickness of concrete, or in slabs at least 1 cm. Only for producing an increased security against fire (see p. 5), is a greater depth taken. † (2 to 4 cm).

At alternate crossings both kinds of rods are bound together by wiring, so that as a whole they form a steel network. The

ends of such wires (0.7 to 1.0 mm diameter) after winding are twisted together with wire pliers.

p. 67 If a floor slab be treated as a beam on two supports and is uniformly loaded, the arrangement of the ~~slab~~^{steel} reinforcement is made according to the following points of view; the bending moments at the supports = 0; they increase regularly toward the middle, there attaining their maximum value; in other words, the compressible stresses in the fibres lying above the zero line as well as the tensile stresses in the lower zone of the slab always increase toward the middle (of span).

Fig. 71 shows the simplest form of a reinforced concrete slab. The reinforcement is only placed in the tension zone, wherefore all compressible stresses must be received by the concrete. It is naturally advantageous to place the steel rods as near as possible to the most strongly stressed external fibres, to increase the useful effect. To save steel, one may proceed according to the arrangement b, thus making the second or third rods shorter. -- If one desires to save concrete, its basal X form in Fig. 72 is chosen. The steel rods here not only lie in the tension zone, but they also aid the concrete in receiving the compressible stresses. Yet the effect of the steel reinforcement in the compression zone is quite unimportant, in comparison to its effectiveness in the tension zone, as will be later proved by calculations. ¹ The dimensions and arrangement of the compression reinforcement naturally does not require to correspond to the tension reinforcement, as Fig. 72 shows in both sections. -- It is advantageous as a reinforcement as in Fig. 73; indeed in regard to the fact, that on the one hand all the tension rods are no longer necessary at the support, but on the other that with the least degree of fixing negative moments may occur. ¹

Note 1. p. 167. See Section IX.

Note 1. p. 168. Fixing results from the more perfect connection of the floor and wall, and from the arched filling of the angle. (See B & E. 1909. p. 295).

The utilization of the structural materials (especially for floors of greater spans) is more efficient, when the slabs are so formed as to become fixed at both ends. First of all, the deflection is then less than with free supports at the ends. The elastic curve shows two points of change of curvature, so that the bending moment at the two points attains the zero

value. Therefore one has positive and negative moments, the maximum value of the former lying at the middle, while the negative are maximum at the fixed ends. Accordingly in the middle part of the slab the lower fibres are in tension and the upper are in compression; conversely at the fixed ends the tensile stresses lie in the upper zone and the compressible stresses in the lower. According to Fig. 75:--

$$+ M_{\max} = \frac{Q l}{24} \quad - M_{\max} = \frac{Q l}{12} \quad \times \frac{8}{5} = \text{about } \frac{1}{5} \cdot \frac{2}{5}$$

But these values require a uniformly distributed loading and are only valid when the cross section of the slab remains unchanged, ² and also the bearing rods suffer no alteration in dimensions and position. But since both these are excluded in most cases, the given values are only an approximation. To save the statical proof of an existing fixing, one can dimension the slab for a maximum moment $M = + \frac{Q l}{12}$ to $+ \frac{Q l}{16}$, taking into account the occurrence of negative fixing moments by arranging arches and by carrying the reinforcement at ends as close as possible to upper edge of slab.

Note 2. p. 168. See "On the Fixed Beam with Variable Moment of Inertia." Ann. B. 1909, p. 171. -- But experiments of the German Commission (Heft 18) furnish evidence, that beams fixed at both ends, as well as beams fixed at one end and freely supported at the other (Fig. 78) may very well be calculated according to the usual mode of calculating homogeneous elastic beams.

It is mostly advisable to reject a complete fixing of the slab, since in many cases the necessary conditions are wanting. Indeed the fixing produces a substantial economy in materials, and an important stiffness of the whole, yet in external walls, so far as not themselves constructed of concrete, regular fixing is made with difficulty. ¹

Note 1. p. 168. For example, no perfect fixing can be asserted, if the slab is only later inserted into a recess. Also see "Fixed Floors and Beams of Reinforced Concrete." Z & B. 1908. p. 116, 215; 1909. p. 290, 381. -- But experiments of Emperger with fixed beams (Report of Austrian E.B.A. 1913) have shown, that beams between walls must always be reinforced for a partial fixing, even if scarcely loaded at the supports, thus being but slightly built in.

If floors are tamped between I-beams and rest solidly on the flanges at both sides, then according to ministerial decree,

they may be calculated only by the formula, $M = + \frac{Q l}{10}$. But in the case, where the junction is arched beneath, and the reinforcement is anchored fast in the adjacent panels, doubtless a higher degree of fixing may be used in the calculations. ²

Note 2. p. 169. Floor slabs between I-beams must in no case be calculated as continuous beams, according to the Prussian Regulations. This calculation is only allowable, if the slabs rest on reinforced concrete beams. For reinforced concrete construction in general must this regulation be regarded as favorable; it directly favors a greater use of uniform arrangement of T-beams.

Slabs extending between two beams as in Fig. 74, may be calculated with $\frac{Q l}{10}$, and only with $\frac{Q l}{15}$, if greater height of beam ($n =$ or more than 4 d) and the usual distances exist between supports.

If one is in position to obtain the points of change of the elastic line, thus to establish where in the upper zone tensile stresses cease and the compressile stresses begin, then may the basal form be employed, as in Fig. 75 a. Only a single steel rod is arranged, but this in consequence of its bending to correspond to the elastic line sufficiently resists both the tensile stresses in the upper zone as well as those in the lower zone. The arched ends afford a better fixing. ¹ If for complete fixing no arches are arranged, then on account of the great fixing moment is required very strong steel reinforcement at the fixing points, but which must extend still beyond the points of zero moments. (Fig. 75 b). -- If no perfect fixing exists, but only a partial fixing, then according to the degree of fixing (Fig. 75) -- only a portion of the reinforcement is carried upward. According to Fig. 76 left, each fourth rod extends into the tension zone, in Fig. 76 right, each third rod. Besides according to Fig. 76 right, above all continuous tension rods a are also placed special rods c in the upper zone. The basal form in Fig. 76 is representative of most floor constructions in reinforced concrete. ¹ For the arrangement of the reinforcement is always determinative of the form of the moment area, above all the rods must not be bent too soon (Fig. 73), the proper reinforcement being dotted. -- According to Fig. 77 the rods c are carried through from one fixed end to the other.

Note 1. p. 170. Every arched end increases the cross section

at the fixed end. The effective depth $h - a$ becomes greater, so that in a given case, in spite of the calculated greater negative moment for fixing, there can suffice the reinforcement for the positive maximum moment (at the middle of the slab). (See Section VIII).

Note 1.p.171. For very thin fixed slabs, however for practical reasons, upward bends are less usual. Here arrangements $\pm$ like Fig. 75 b are preferable.

Fig. 78 shows a beam with one end fixed and the other freely supported. The forms of the reinforcement here correspond to the occurrence of tensile stresses in the lower zone at the supported end, and in the upper zone at the fixed end.

With structural members projecting as consoles, slabs are to be regarded as fixed at one side. The stresses in the building materials are opposed to those in simple beams on two supports; the upper fibres are in tension and the lower in compression, for which reason the upper half of the cross section is to be strengthened by steel reinforcement, as shown in Fig. 79. -- If the projection is quite unimportant in comparison to the cross section of the slab, then it is advisable also to place steel rods in the compression zone.

Fig. 80 shows a freely supported beam with an overhanging end. (Compare this with Fig. 76). An arched arrangement as in Fig. 80 b permits a reduction of the tensile reinforcement at the point of negative moment of the support. (Greater height $h - a$).

On steps of artificial stone with fixed end see Part II, 7th edition, p. 65. (Decree of Berlin Building Office. March 19, 1913).

Concerning the minimum thickness of slabs, the Prussian Regulations prescribe the following:--

"The calculated thickness of slabs and of slab-formed parts of T-beams is everywhere to be made at least 8 cm."

For the work of many inexperienced contractors may this new rule be of a certain use; for any even small deviation of the steel rods from their calculated height may have the most injurious effect on the general construction, particularly if occasional snocks are not to be avoided. Yet in many cases both raw materials are not utilized with sufficient economy. For slabs for reinforced concrete roofs and hall buildings a smaller thickness (to 6 cm) is still permissible.

smaller thickness (to 6 cm) is still permissible. For aslar and hollow block floors in the form of T-beams with the smallest possible subdivision by ribs (30 to 50 cm), as well as for suspended plastered ceilings, the regulation does not come in question; also see Section XIV (Decree of Nov. 22, 1913).

Floor slabs in simple buildings are not generally made thicker than about 12 to 14 cm; otherwise is advisable rather the use of ribbed floors (T-beams). For cantilever floors, one may go to 20 cm in thickness; but otherwise here also mostly T-beams are more economical. Only for the driveway floors of bridges does one go to the thickness of 30 and 35 cm. One must always consider, that the dead weight increases in an important degree with the increasing depth of the slab. ¹

Note 1. p. 173. A reduction of the dead load is obtained by the use of hollow blocks (see Section III h) or of pumice concrete floors (see p. 45).

Notable is also the requirement of the Austrian Regulations, according to which slabs with less than 6 cm least thickness cannot be assumed in calculations as cooperating in case of T-beams.

Also see in Section VII a the loading test given for a reinforced concrete floor only 4.5 cm thick.

C. Slabs reinforced Crosswise.

Rooms of rectangular and approximately square form may advantageously be covered by slabs reinforced crosswise, which are freely supported at all sides, are half or completely fixed. The assumption of support at all sides affords a certain economical advantage, since here the otherwise usual distributing rods aid in the supporting effect. The advantage of less concrete is indeed opposed to the disadvantage of the greater use of steel.

The Prussian Regulations prescribe the following:--

"Slabs supported at all edges and furnished with crossed reinforcement, for a uniformly distributed loading, if their length a be less than $1 \frac{1}{2}$ times their breadth b , may be calculated by the formula $M = \frac{p}{12} b^2$. Against the negative fixing moments of the supports are to be provided arrangements by the form and position of the steel rods."

Here also is considered but one dimension of the slab (the shortest side); both for the square as well as for the recta-

rectangular slab -- with equal length of the side -- the same value is used, and the cross section found for the steel is τ then to be placed lengthwise and crosswise the slab. This official rule presents few advantages; in all cases the cost of such slabs under some conditions would be higher than for simply reinforced slabs. One would do better to arrange intermediate ribs, producing less thickness of slab and more favorable distribution of the loading. The technical advantage of a crossed reinforcement, obtaining a more uniform transmission of forces, and the aid of the distributing rods in the statical cooperation, is however lost.

The difficulty in the calculation of crosswise reinforcement of slabs lies first in the indeterminant forces at the supports in reference to magnitude and distribution along the edges. For example, for a corridor floor of slabs may rest freely on the walls, but are firmly fixed at the beams (Fig. 81). In this case must one certainly take into account the fact, that in the direction of the walls the slabs are substantially stiffer than in the other direction, in consequence of this fixing. ¹

Note 1. p.174. B & E. 1911, p. 243, 302 (with a comparison of different official regulations).

The effect of crossed reinforcement is so much the greater, the more nearly the plan of the room approximates the square. It will also be advisable, to make the middle of the slab especially strong, since there the stress reaches the maximum value. If in Fig. 82 L be the longer and l the shorter side of the rectangular room, then one employs with a uniformly distributed load the following relations for calculating the bending moments at the middle of the slab:--

$$M_L = \frac{Q L}{8} \times \frac{l^4}{L^4 + l^4} \quad ; \quad M_l = \frac{Q l}{8} \times \frac{L^4}{L^4 + l^4} \quad 2$$

Note 2. p.174. Derived from the equation for deflection of slabs with fixed edges and uniformly distributed loading.

$$\delta = m \times \frac{Q l^4}{E_b I} = m \times \frac{P L^4}{E_b I}$$

Here is assumed a free support at all sides. If a certain fixing moment is to be considered, then one introduces the formulas $\frac{Q L}{10}$ and $\frac{Q l}{10}$, or $\frac{Q L}{12}$ and $\frac{Q l}{12}$ (according to degree of fixing).

With complete fixing as in Fig. 83 (strongly arched junction and continuous slab) is to be inserted in the formula:

For middle of slab $\frac{Q L^2}{24}$ and $\frac{Q l^2}{24}$.

At the support $-\frac{Q L^2}{12}$ and $-\frac{Q l^2}{12}$.

For a freely supported square slab, $L = l$, thus $M_L = M_l = \frac{Q l^2}{16}$.

Note 1. p. 175. According to the investigations of Pöppl the moment for the freely supported square slab is $M = \frac{Q l^2}{24}$. For practical purposes can be taken the average value $M = \frac{Q l^2}{20}$, perhaps for complete fixing, $M_m = +\frac{Q l^2}{30}$ at the middle of the slab, and $M_a = -\frac{Q l^2}{20}$ at the edge.

According to the views of Professor Mörsch, for square slabs uniformly reinforced in two directions, one proceeds more safely, if for the positive and negative bending moments are each taken one half those which result for a continuous beam extended in one direction.

In the following Table are collected the values of $\frac{l^4}{L^4 + l^4}$ and $\frac{L^4}{L^4 + l^4}$ for the most common proportions of rooms. The crosswise reinforcement exerts no effect worth mention, as soon as $L = 1.5 l$. From this limiting value onward the double sided support is to be neglected, and the moment obtained in reference to $\frac{l^4}{L^4 + l^4}$.

Let $\frac{L^4}{L^4 + l^4} = \alpha$; $\frac{l^4}{L^4 + l^4} = \beta$.

$\frac{L}{l}$	1.0	1.1	1.2	1.3	1.4	1.5
α	0.500	0.592	0.675	0.740	0.793	0.835
β	0.500	0.408	0.325	0.260	0.207	0.165

For example if $L = 5.0$ m, $l = 4$ m, $g = 300$ kil/m², then by use of the preceding Table results the following calculation.

$\frac{L}{l} = 1.25$: $\alpha = 0.710$: $\beta = 0.290$ (by interpolation).

$$M_L = \frac{300 \times 5^2}{8} \times 100 \times 0.290 = 54,375 \text{ cm kil.}$$

$$M_l = \frac{300 \times 4^2}{8} \times 100 \times 0.710 = 85,200 \text{ cm kil.}$$

The moment M_l about the shorter side determines the thickness of the slab. The reinforcement itself will be placed closer next the middle of the slab for practical reasons, than at the edges (Fig. 84).

On the basis of experiments made by Bach (also see B. & E. 1909, p. 3836: 1910, p. 130) and which have shown that the

maximum bending moment must occur in the direction of the diagonals, that it is also very probable that the diagonal section is the dangerous cross section, the formula was obtained:—

$$M_{\max} = \frac{p}{12} \times \frac{L^2 l^2}{L^2 + l^2} \text{ and for square slabs, } M_{\max} = \frac{p L^2}{24}.$$

It is here assumed that the loading is uniformly distributed on all four sides, one fourth to each. ¹

Note = p. 176. Strictly taken however, the pressure on the supports for crosswise reinforced slabs is concentrated at the middles of the slabs. One is here justified in the assumption, that with a continued increase in the loading the slab will rise from the supports at the four corners.

RV 27 The Swiss Regulations (likewise the Württemberg Regulations) assume the total force equal to the sum of the supporting forces of two separate simply reinforced slabs, and recommend the distribution of the loading according to the following relations.

$$\text{Portion of loading } p_l = \frac{L^2}{L^2 + l^2} p \text{ for the span } l.$$

$$\text{Portion of loading } p_L = \frac{l^2}{L^2 + l^2} p \text{ for the span } L.$$

It is here assumed, that the supporting force depends on reaching the elastic limit in the steel reinforcement.

The Austrian Regulations prescribe the following requirements.

$$\text{Portion of loading } p_l = \frac{L^2}{k \frac{l^2 + L^2}{k} l^2} p \text{ for the span } l.$$

$$\text{Portion of loading } p_L = \frac{l^2}{k \frac{l^2 + L^2}{k} L^2} p \text{ for the span } L.$$

Here k denotes the ratio of the sectional area of the series of reinforcements parallel to L to the sectional area of the series of reinforcements parallel to l , both sectional areas referred to one lineal m. Corresponding to this distribution of the loading are to be obtained the shears, loads on supports and bending moments. The calculation only has to be made for the least side, as soon as the sectional area of one series of reinforcements amounts to less than 30 p.c. of the sectional area of the other series of reinforcements. For continuous slabs one must proceed properly according to the mode of calculation. For all continuous or fixed slabs, however, the positive panel moments must be taken at not less than $\frac{2}{3}$ of the panel moments resulting for a freely supported slab of a single panel. ¹

Note 1. p. 177. Besides the literature concerning crosswise

reinforced slabs, reference is made to the following publications:--

Bosch, Calculation of crosswise reinforced concrete slabs. Wm. Ernst & Son. Berlin. Danusso, Lecture on calculation of a crosswise reinforced concrete slabs. Wm. Ernst & Son, Berlin. B & E. 1905. p. 177; 1910, p. 130, 355; 1911, p. 243; 1912, p. 37. Contribs. 1909. p. 62, 87, 74, 92; 1912, p. 1, 162, 174. "Calculation of beams under crosswise reinforced slabs," Z & B. 1910. p. 311, 327.

d. Continuous Slabs and Beams.

Fixed and continuous slabs and beams play substantially a greater part in reinforced concrete construction than in pure steel construction, where the fixed ends are always less so, assuming similar conditions.

The Prussian Regulations prescribe the following:--

"For slabs and beams, that continue over several panels, in case the actually occurring moments and reactions at the supports cannot be calculated according to the rules in force for continuous beams under the assumption of free support on the middle and end supports, or cannot be proved by experiment, the bending moment at the middle of the panel must be taken at $\frac{1}{5}$ the value, that would exist for a slab freely resting on two supports. Above the supports is then to be assumed the negative bending moment as large as the bending moment in the panels on both sides with both ends freely supported." According to this rule, slabs and beams can only be taken as continuous, if they everywhere rest on firm supports lying in a plane or on reinforced concrete girders. In the arrangement of the reinforcement in all cases particular care must be taken for the possibility of the occurrence of negative moments.

For beams a fixing moment at the ends can only be assumed and employed in calculation, when particular structural arrangements afford an assured fixing.

The assumption of continuity for calculation must not extend over more than three panels. For live loads of more than 1000 kil/m², the calculation is also to be made for the most unfavorable distribution of the load." ¹

Note 1. p. 178. But according to the decree of April 11, 1908, the calculation for live load uniformly distributed over the separate panels is not made generally, and also is not permitted for loads of less than

permissible for loads less than 1000 kil/m². Otherwise one might particularly place the moment at middle of panel $= \frac{Q \cdot l}{40}$, for p less than 1000 kil/m², corresponding to the actual wording of the preceding Regulation. But an unequal distribution of the ^{live} loading much more corresponds to the actual conditions in dwellings and business buildings than a uniform distribution.

On the basis of these requirements, assuming uniformly distributed loads g and p and uniform spans l, the method of calculation given on page 179 is possible.

As supports of continuous slabs or beams may serve:--

- Walls or supports of concrete or of masonry.
- Girders of reinforced concrete (T-beam ribs), or I-beams.

Moments.		Approx. calc-n. $\frac{10}{8} \text{ and } \frac{Q \cdot l}{8}$	g uniform, p variable. Exact calculation.
2 pan.	Pan. Moment	$+ 0.1 (g + p) l^2$	$+ (.07 g + .095 p) l^2$
3 sup.	Sup. moment	$- .125 (g + p) l^2$	$- (.125 g + p) l^2$
3 or more			
pan	end.pan.M.	$+ 0.1 (g + p) l^2$	$+ (.03 g + .10 p) l^2$
4 or	mid.pan.M	$+ 0.1 (g + p) l^2$	$+ (.025 g + .075 p) l^2$
more		and	$+ (.025 g + .05 p) l^2$
supp.	sup.mom.M	$- .125 (g + p) l^2$	$- (.10 g + .117 p) l^2$

(See Table in Appendix).

For the walls or supports for the same girder are employed as far as possible the same building materials and similar foundations (uniform pressure on the soil): cracks in a continuous girder are very often to be referred to the fact, that partly free support on masonry, and partly fixing to slender and elastic reinforced concrete supports are employed. Particularly at the middle supports sooner show a tendency to sink, than the external walls.

Slabs on reinforced concrete ribs (T-beam floors), according to previous experience may unhesitatingly be calculated as continuous beams over supports of equal heights, so far as the free lengths of the ribs are kept within the usual limits. The deflection of the ribs is only small, particularly if these be again arranged as continuous. Likewise a very stiff connection between slab and rib exists. For greater spans of beams is advantageous the arrangement of load distributing cross beams, that permit no great differences in the elastic deflection of

of the adjacent beams. On the contrary, if the ribs consist of I-beams, then must primarily a greater deflection of the beams be counted on, that results in an unequal height of the supports of the slab. Besides, the connection between the I-beams and slabs cannot be as free from objections as with reinforced concrete beams. On these grounds it is generally improper to calculate slabs resting on I-beams according to the rules for continuous slabs or beams. One is here satisfied, according to the construction, with about $M = \frac{Ql}{15}$ for the middle and about $\frac{Ql}{10}$ for the end panels, as well as a sufficient reinforcement at the locations of the negative moments at the supports. For unequal panels negative moments may also come into consideration. (See p. 182, under 5). ¹

Note 1.p.180. Further see B & E. 1913. p. 435.

Also see the Regulations of the President of the Building Office of Berlin concerning ground principles of the calculation and construction of reinforced concrete T-beam floors, Decree of Nov. 22, 1913. (Section XIV of this volume).

For calculating continuous slabs and beams in reinforced concrete construction, the following should be noted:

1. As for approximate calculations, then $M = \frac{Ql}{10}$ produces very deep sections for the middle panel, but tolerably accurate ones for the end panels. The use of the Tables for the maximum moments (see Appendix) affords substantial economy in materials. Otherwise for approximate calculations the following values are to be recommended, but only for small spans of slabs and small live loads. (See Fig. 85 for this).

M_g (over intermediate supports) $= \frac{Ql}{10}$, but $\frac{Ql}{8}$ for three spans. (For simple floor slabs, on account of the arched connection and the bending of the steel, one can generally omit a special calculation of the Moment - M_g of the support: also see Fig. 163.).

M_z (in the intermediate panels) $= + \frac{Ql}{15}$ to $\frac{Ql}{18}$.

M_e (in the end panels) $= + \frac{Ql}{10}$ to $\frac{Ql}{12}$.

2. It would be advisable to extend the calculated assumption of continuity over more than three panels: for the moments M_g and M_p of the middle panel become so much the greater, the more panels that exist. Only for the determination of the sections of the end spans suffices the assumption of continuity for three spans.

three spans. The maximum values for moments and shears are found for partial-loading, according to Fig. 86. ¹

Note 1.p.181. Also see Boerner, Statical Tables. 5 th edition. p. 101.

3. If the live load p is assumed to ~~be~~ ^{be} uniformly distributed over the entire beam, for example as usual for roofs with uniformly distributed snow load, then the moments of the middle panels would be too small: for example, for four supports the moment in the middle panel is only $M = \frac{(q + p) l^2}{40}$. Already on practical grounds, one should not go below the value for complete fixing, $M = \frac{(q + p) l^2}{24}$; for to the saving in materials is opposed a reduction of the factor of safety. ²

Note 2.p.181. It may occur in test loadings, that by loading only a strip of the middle panel (thus without loading the end panels), the allowable stress in the concrete at the centre will exceed more than fivefold, and that the tensile stress in the steel reaches the tensile resistance, even if not quite exceeding it. -- It is also further proposed, to only fully load the panel to be tested, assuming the other panels as only loaded by their dead weight, but this may lead to false results. See B & E. 1911. p.88.

4. If greater live loads are to be taken, then even with a division in equal panels a continuous upper reinforcement is ^{to be} recommended. For smaller panels under some conditions, a greater thickness of slab is necessary. (Fig. 87). From $p = 800 \text{ kil/m}^2$ is advisable an exact investigation of the existence of negative panel moments. But to be considered is also the frequently important reduction of the panel moments by resistance to rotation of the strong, rigidly enclosed reinforced concrete girders. (Fig. 88).

5. Negative panel moments (great live loads, unequal distribution on the panel) are especially to be considered for beams. According to Fig. 39 must even be assumed a greater depth of beam in the middle panel, since otherwise it would lack the required compression section (below). In extreme cases, to reduce the depth of the beam one may extend the width of beam for the middle panel or the width of support as in Fig. 89 b. One then rather has to do with a portal beam (stiff frame). To be considered is, that by the stiff arched connection of the middle panel beam with the support the negative moment is materi-

materially reduced. The mathematical proof is laborious, but it may afford considerable saving in cost. The supports are ~~then~~ also to be investigated for bending. ¹ (see Figs. 90, 91).

Note 1. p. 183. The bending moment, which the beam in consequence of its rigid connection with the support exerts on that, is so much the greater, the larger the moment of inertia of the section of the pier is in comparison to that of the section of the beam. For slender (for example spirally banded) columns a calculation is more necessary than for thick columns. (See among others B & E. 1912. p. 271. Further, Scheidt-Probst, "Investigations on continuous reinforced concrete structures." Berlin. 1912. (Z. B. 1912. p. 194).)

6. The so-called Winkler's numbers given in the Appendix for the calculation of continuous slabs assume freely movable pointed supports, but which indeed never exist in practice. In consequence of the fixing at the ribs or supports, the reciprocal influence of the adjoining panels is less: also the girders oppose a great resistance to rotation. For beams with more than five supports the middle panel ^{may} be approximately calculated as the second panel, and the end panel as the first panel of the beam on five supports. Thus one restricts himself not too strongly to the moment areas, and for example, he always assumes a continuous tensile reinforcement, even at the supports, where it is properly not necessary. -- The Tables properly require a beam construction with constant cross section and constant moment of inertia, which however is excluded in consequence of the varying position and size of the reinforcement in reinforced concrete. The Tables are also only valid for equal or but slightly varying spans. If greater differences in spans exist, then the calculation is best made by the aid of the equation of three moments (Clapeyron's equation) ² or by the graphical method according to Ritter. ³ By the use of supports ~~one is dependent on their building materials, but especially~~ on the reliability of the foundations and of the ground at the building. For example, in a certain case ($p = 600$, $g = 700$ kil/m lineal), according to Fig. 92 for a difference in height of the supports of only 2 cm, the shears increase about 55 p.c., and the moments even 165 p.c. A sinking of the support always reduces the moment at the support, until it becomes zero and finally is even positive. The beam then generally rests on it

p. 186

designate the supports, beginning at the left, then one finds approximately by the aid of Clapeyron's equation, assuming equal spans:-- ²

Note 1. p. 186. For 7 to 9 supports, in the Table for loads on supports D and E are inserted values about 1.0.

Note 2. p. 186. For very small end panels may result negative loads on the supports. In such cases ~~the~~ ^{for} unequal panels exact calculations are naturally necessary.

Supp. No. of supports (load on supports = $K \pm (g + p)$).

	2	3	4	5	6	7	8	9
A	0.375	0.400	0.3929	0.3947	0.3942	0.3944	0.3943	
B	1.250	1.100	1.1423	1.1517	1.1346	1.1257	1.1240	
C	-----	-----	0.3236	0.9736	0.9616	0.9649	0.9640	

According to the Regulations on p. 178 (last paragraph) the calculated assumption of the connection does not extend over more than three panels, then for the load on the middle supports must be inserted $B = C = (1.1 g + 1.2 p)l$, corresponding to a panel moment of $(0.025 g + 0.075 p)l^2$. ³

Note 3. p. 186. Sometimes for the loads ^{on} supports-- in reference to increased live loads -- fixed additions (up to 10 p.c) are prescribed to increase the safety.

For the structural treatment of continuous slabs the following is to be stated:-- Over the points of support the steel reinforcement is to be placed in the upper zone of the cross section and near the external layer, to receive the tensile stresses (see Figs. 95, 94): otherwise the arrangement of the reinforcement is according to the same points of view, as already stated. Accurate representations of the reinforcement of continuous slabs are seen in Figs. 96, 97. Stirrups are generally unnecessary: the bending of certain rods follows, not on account of the shears, but indeed to transfer the lower tension rods of the middle of the slab into the tension zone lying over the supports. The cross section over the supports is generally made so deep, that there the same amount of steel suffices as at the middle of the panel. From the data in Fig. 99 it may be seen, that the continuity of a slab as opposed to freely resting on the supports, may have as a result an important economy of materials in consequence of reducing the thickness of the slab.

p. 187, 188

Note 4. p. 186. But it is always advisable to also allow the ~~reinforcement over the supports~~

reinforcement above the supports also to pass through below, since by the unequal sinkings of the supports (see Fig. 97 d) tensile stresses may also occur below. The lower continuous reinforcement also affords a better connection of slab and rib, more secure against shear.

Fig. 97: reinforcement of a slab extending over three panels. (a). If the panels are equal, then in the middle panel as Fig. 97 a shows, generally steel reinforcement must also be arranged at top (unfavorable effect of the fully loaded side panels). This reinforcement is then saved, if according to Fig. 97 e & the end panels are smaller than the middle panel (at least $l_1 = 0.8 l$). Fig. 97 f shows one slab extending over two panels.

If I-beams are employed for the beams, then may the reinforcement be according to Fig. 97 b. Arrangements according to Figs. 97 c and d are not recommended; the reinforcement must extend into the adjacent panel as in Fig. 97 b. -- Fig. 97 g shows that continuous reinforcement below is entirely proper. (See Note on p. 186).

Fig. 98; different conditions of solid slabs between mild steel I-beams. ¹

Note 1.p.189. More extended explanations of Fig. 98 are to be found in Part II, 7th edition. p.7.

e. T-Beams.

For larger dimensions of rooms are employed the so-called T-beams; parallel to the shorter side of the room are placed beams, and these are connected together by floor slabs (with or without cross beams). If the span be too great, or if one be exposed to important restrictions in the choice of the structural height, then may the T-beams be supported by columns.

The usual section of a T-beam is evident from Figs. 100, 101. Since quite important shears occur between the slab and the rib, the rib thus tending to separate from the slab, the dangerous section being mostly at the under surface of the slab, and when the section is increased by arches with the addition of stirrups, it is made more suitable for receiving such shears. (Fig. 100).

As for what concerns the arrangement of the reinforcement, then for the T-beams, according as they are free or fixed at both ends, the same is true as for simple slabs. Positive moments require the reinforcement as close as possible to the

lower surface of the concrete, and negative moments require the reinforcement as near as possible to the upper surface.

Fig. 100 exhibits different arrangements of the steel reinforcement. Fig. 100 I represents a beam on two supports; to receive the shears serve stirrups and bands in the rods, which should prevent the dangerous shear cracks. (Fig. 67 e, p. 161). To support the bearing rods while concreting are frequently advisable special supporting rods in the compression zone as in Fig. 101 b. -- Fig 101 II shows a beam on three supports (with middle column). Likewise here the rods are shown separately to make the arrangement of the reinforcement more apparent. The connection with the support is arched; for the stiff junction are taken three separate arch rods. A construction according to Fig. 101 c is less recommended for the same purpose -- for purely practical reasons.

According to Fig. 102 the construction of the floor of a rectangular room is in this way, that a main girder A and two side beams B are placed at right angles. The distance apart of the beams B varies greatly, and is particularly arranged according to the dimensions of the interior to be covered. It generally varies between 1.5 to 3.0 m. ¹ The less the distance between beams, the thinner may be made the intermediate slabs; and conversely, thick slabs are necessary when the beams are placed far apart. The width b_1 of the beam mainly depends on the number of rods to be inserted. -- The forms for T-beams require more expense than the centering for the simple slabs; yet the use of a beam floor is almost always economically preferable to that of a continuous slab floor.

Note 1. p. 191. One should not take the distance between the beams too large; economical advantages in reinforced concrete floor construction as a rule only exist, if one uses the same spacing of beams and supports, to which he is accustomed in steel construction.

The supporting of the beams generally is according to Fig. 103; the beam extends into the wall, leaving the slab between the beams to rest on the wall. In many cases it is advisable however to have a support as in Fig. 103, according to which beam and slab end in a continuous supporting wall beam.

On the structural treatment of T-beams see Section X.

f. Vaults.

Vaults in reinforced concrete, besides compressive stresses may also receive tensile stresses, wherefore they can be made thinner than tamped concrete vaults. Their axes may be horizontal, inclined or vertical (ceilings, roofs, bridges and foundations -- stairs and ramps -- retaining walls, tanks for fluids.

For concrete vaults reinforced ~~concrete~~ reinforced with steel occurring in simple buildings, two forms are to be distinguished. The first of these exhibits the vault in both the intrados and the extrados (Figs. 105 a, 106), while for the other principal form only the intrados is vaulted, the extrados being plane, on the contrary. (Fig. 106 b). Vaults of the last kind are much employed, particularly in buildings, and they are therefore to be recommended, since they exert less side thrust on the abutments, and also the construction is simpler and easier. Certainly the weight of such a vault is mostly considerably greater than for vaults with curved extrados, assuming equal loadings and spans.

If small dimensions and loads are to be considered, and the arch form approximates the parabola, then suffices for the two principal forms a reinforcement in the intrados zone (Figs. 105 a, b). For medium and larger spans is advisable a reinforcement, both in the intrados as well as the extrados zones. $\bar{T}$ (Figs. 105 d, e). Such doubled reinforcement also corresponds to the occurrence of transverse cracks, which have appeared in both zones with a live load on one side of the vault. Instead of the continuous reinforcement of the extrados, there suffices under some conditions an anchoring of the extrados at the abutment, as in Figs. 105 f, c). The intrados and extrados lines may be parallel (Figs. 105 a, e); yet particularly for larger spans a deepening of the vault toward the springings is generally more advisable. (Fig. 105 a). -- According to Fig. 107 the vault is formed into ribs and these are connected together by closing ceilings like T-beams.

Since now in very few cases cracks parallel to the axis of the vault are observed, the addition of stirrups to receive any existing shears is generally unnecessary in simple buildings. But a strengthening by stirrups is to be recommended in Fig. 105 d, e.

Always to be considered is a complete unyielding of the abutment; ever so small a yielding of this to the existing horizon-

horizontal thrust (especially for small rise) may result in the greatest dangers to the permanence of the vault.

g. Supports.

For reinforced concrete supports are generally required the least possible sectional dimensions. If a concrete member of square, rectangular or round section (most common is the square section), there are near the external surface continuous vertical steel rods, connected together at distances of 20 to 40 cm by transverse connections -- wire bands (Figs. 51, 110). Between the external surface of the concrete and the reinforcement must be a sufficient distance, so that the concrete is in condition to prevent bending and rusting of the rods, as well as to afford an effective protection against the heat of a fire. The reinforcement mostly consists of round rods, which are symmetrically distributed in the cross section, and are arranged parallel to each other. -- For continuous supports, to make easier the erection of the building, the butt joints in the longitudinal rods are not placed in the lower column (Fig. 108 a but in the upper column, as in Fig. 108 b; also see Fig. 101. -- An example of treatment of the base is shown in Fig. 109. (Also see Fig. 19 on p. 72).

Further see Section XV as well as Part II, 7th edition, Section I B, p. 47 to 53.

On concrete columns spirally banded with steel, see Section XVII.

Fig. 110 exhibits the general arrangement and the distribution of the steel for a larger floor design, consisting of floor slabs, main and cross beams and supports. The floor rods are carried over the beams and are hooked together there. ¹ Likewise several rods of the beam are carried above the supports ¹ and anchored in the adjoining panel. Main and cross beams show stirrups, whose ends are bent (see Fig. 100). The supports are reinforced as in Fig. 101 and are furnished with cross bands.

Note 1. p. 195. -- Such an anchoring of the steel however, is not to be recommended. See Fig. 97 c and p. 188, last lines.

n. Various treatments of the usual forms of floors. (Fig. 111) ²

Note 2. p. 195. For further data on these, see Part II, 7th edition. p. 5.

- a. Simple solid slab fixed at both sides.
- b. Simple solid slab freely supported.

- c. Vaulted solid slab.
- d. Vaulted solid slab with plane top surface.
- e. Ordinary T-beams with visible ribs.
- f. Ordinary T-beams with suspended Rabbitz ceiling.
- g. Brick and steel slab (reinforced brick floor) with and ~~without~~ without compression slab.
- h. Ribbed slab with inserted light bricks and a visible concrete ceiling.
- i. Ribbed slab with inserted light bricks laid directly on the centering.
- j. Ribbed slab with hollow blocks on plane concrete ceiling.
- k. Ribbed slab with inserted hollow blocks.
- l. Ribbed slab with inserted hollow blocks laid directly on centering.
- m. Ribbed slab with inserted hollow blocks with projections below to avoid any concrete ceiling.
- n. Ribbed slab with ribs remaining visible (hollow slabs); the (sheet metal) forms are removed.
- o. Slab ceiling of beams delivered ready for use, without a and with separate compression layer.
- p. Slab ceiling as before, but with hollow blocks lying between it and a later concreted compression layer.

1. Some Rules for Graphical Representation of Reinforced Concrete Buildings.

The drawings must be in line as in description clear and sufficiently plain, ¹ to show all details of the reinforcement. Superfluous dimensions are to be omitted for sake of greater clearness. In particular cases, the building officials will require a drawing of the forms intended, as well as drawings for unusual and strongly stressed structural parts, and also for methods of construction protected by patents.

Note 1. p.186. Important is the regulation by the President of the Building Office in Berlin, of Dec. 19, 1912.

"In recent times have been repeatedly been brought to me statistical calculations and explanations for certain architectural constructions for checking, on which the stamped notes are not clear or were already faint, or on which the lettering with a calline or ink pencils could not be read by artificial light on account of its gloss. Generally the written portions were on loose sheets or only held together by loose angle clips, and produced without numbers.

Since such writings make very difficult the regular treatment in the business procedure of the Building Office, or are even entirely unsuitable, I have directed them to be returned immediately. Therefore I request owners and contractors in their own interests, to only bring such documents, that do not present these defects. "

For simpler buildings are first necessary ground plans, if possible at the scale of $1/50$. The cross sections of reinforced concrete floors are to be drawn in black (without light edges), stating the thickness, the subdivision by ribs, the reinforcement per m. width of floor, etc. Separate drawings for the ordinary floor slabs are generally unnecessary; here suffice inserted sections as in Fig. 112 a. The underside of the floor is drawn, thus all ribs are in full lines (vault lines ^{p. 197, 198} finer, see Fig. 112 e). The steel rods are drawn separately. But for the structural superintendence costly suffice simple representations as in Fig. 112 b; above the floor is given the thickness of the concrete, below it is the reinforcement necessary for the middle of the panel. A similar method of indication is shown by Figs. 112 e, f. In the illustration last mentioned, the dimensions of the arches are also given, and in Fig. 112 a, also the distributing rods. Fig. 97 presents drawings for floors at a greater scale with separate steel rods. If a particularly accurate drawing is required, several sections may be drawn as in Fig. 113. ¹

Note 1. p. 198. But in general to draw the steel separately as in Fig. 97 is more common and more suitable.

For the girders separate drawings are generally necessary, to show all rods, distances between stirrups, bends, reinforcement by stirrups, oblique rods at connections and heights. An example of this is shown by Fig. 100 (p. 190). All rods are drawn full, therefore not hatched (one regards the bearing as made of glass). To obtain greater clearness for the bearing rods, according to Fig. 114 k, the reinforcement is only drawn for one-half; also see Fig. 101, II, for this. In Fig. 112 a, b, e, f, dimensions and rods of the girder are also given. For example, 30 X 48 signifies that the width of girder is 30 cm, height to top of concrete floor being 48 cm. In Fig. 112 f is also given what steel is to be ^{bent and} found what is to remain straight. In the special drawings according to Fig. 114 i, one can give sufficient clearness for the bending by corresponding

figures. The dimensions required for a T-beam section are shown by Fig. 112 g.

According to Fig. 112 n the floor and beam reinforcement are to be shown separately. If one draws separately for both cases the rods, ¹ the clearness of the representation leaves nothing more to be desired. It is always advisable to place the lower tension rods below the drawing of the section, on the contrary the rods of the upper zone being placed above it. (Fig. 112 a).

Note 1. p. 199. The example in Fig. 112 n is graphically executed in all details in Part II, 7 th edition, p. 111.

Representations of supports and of window lintels are presented in the plan, Fig. 112 c, d.

Concerning the data of steel reinforcement in the separate drawings, reference is made to Fig. 114 a, b, d; with a varied reinforcement of the beam accurate drawings are indispensable. (Fig. 114 d). Between the bearing and distributing rods (shown in section) are left a small space (Fig. 114 a). The sections of the steel require no light edges, but always the middle axis (for laying off the dimensions by the dividers). Diameters of steel are given in mm, all other measures being in cm.

Stirrups for beams can be indicated as in Figs. 114 b, c, g; double lines (c) are advisable only for details at larger scale (for example 1 : 5) or for architectural drawings at full size. The same is true for the wire bands for reinforcing supports; Fig. 114 e gives examples at large and small scales. (See Fig. 101).

Graphical illustrations of the end nooks and bends of the bearing rods are presented in Fig. 114 f. The rounding of end nooks is recommended particularly if the rod ends within a panel, thus between other continuous rods; with the incorrect representation given in Fig. 114 f, one does not know whether the ending rod comes from the left or right.

Of particular importance is the tabulated bill of steel; it must contain diameter, number, length to be cut, sort number (location) and sketches with lengths as in Fig. 114 l. An example of such a Table is shown in Fig. 115. ¹ According to this bill of steel then follows the order for the work and the bending of the steel. In the steel bill must also be noted the stirrups with all requirements for the Table (thus also including sketches and number of pieces)



Section VII. Strength, Allowable Stresses, Results of Experiments.

The Official regulations of the different countries in reference to the allowable values of resistance are quite different. (See Table on p. 224, 225). They give in part fixed limiting values; but partly the strength is made dependent on the chosen mixing proportions, on the degree of vibrations to be expected for the structural member concerned, etc. Accordingly it must always be the aim to create structures with a sufficient degree of safety, all the same whether the dead load is great and the live load small, or conversely. Fixed mixing proportions are not required, but definite strengths. (See p. 67).

Basal for all determinations are, in addition to the separate experiments of the specialists:--

a. Publications of "German Commission on Reinforced Concrete." (See p. 3). Wm. Ernst & Son. Berlin. A total of 29 Hefts up to May, 1914.¹

Note 1. Hereafter designated by D. A.

b. "Researches in the Domain of Reinforced Concrete." (See p. 3). Wm. Ernst & Son. Berlin. Total of 24 Hefts until May, 1914.

c. "Contributions on Researches in the Domain of Engineering." Published by Union of German Engineers. J. Springer. Berlin. In these contributions appear the Reports of the Commission on Reinforced Concrete of the Jubilee Foundation of German Industries.

d. Contributions on Experiments made by Commission on Reinforced Concrete of Austrian Union of Engineers and Architects."

a. Resistance to Crushing of Concrete and of Reinforced Concrete.

The principal means of judging of concrete is formed by its resistance to crushing. Crushing forces always exert a bursting effect on the concrete. Most official regulations here make a difference between compression by flexure and that by purely normal crushing. The former is almost without exception higher than the purely normal compressible stress allows. In the rule it is prescribed, that the degree of compressible strength of a concrete shall be determined by the so-called resistance of a cube.¹ This is dependent upon the combination of the raw materials, the goodness of the crushed stone, the mode of mixing (machine mixing gives higher values for resistance (see p. 136,

Note 3), as a rule, crushed stone gives a higher resistance to crushing than coarse gravel, as well as sand of mixed sizes of grain. ² Favorable is also a reduction of the water added. ~~to~~ (See p. 40). ³

Note 1. p. 203. By resistance of a cube (see p. 120) is to be understood the crushing resistance of a cubical mass. The numerical value of the resistance depends on the form of the test block, especially on the ratio of its sides to its height. If this ratio be small (mortar joints), then the crushing resistance is high; in the converse case ~~and~~ the fracture results ~~if~~ from exceeding the resistance to shear. See the Essay of Graf, "Results of Experiments on the Crushing of Concrete cubes; collection of results. Arm. B. 1914. p. 197, 243.

Note 2. p. 203. See page 37, 43.

Note 3. p. 203. According to experiments of Brabandt on the resistance of concrete, the most favorable addition of water lies between 15 and 17 p.c. of the volume of cement and sand.

Fig. 117 gives the graphical representation of the average results of gravel concrete (28 days) in the working years 1907 to 1910, obtained at the Materials Testing Laboratory at Gross-Lichterfelde. Cement content and resistance pretty regularly diminish with increasing content of aggregate in the concrete, i.e., with increasing leanness of the mixture. The average value of ~~the~~ 200 kil/cm² resistance of the mixture 1 : 5 is the mean of limiting values from 106 to 310 kil/cm² (All cubes were made in the testing laboratory, and according to the standards for comparative crushing experiments with tamped concrete. -- In the diagram (Fig. 117) is also included a comparative line for the average values of the resistance to crushing of gravel concrete mixtures, such as may be expected from proper making of concrete and the use of the best raw materials. ¹

Note 1. p. 204. As proof of the possible variations for the resistance values in crushing tests, those by the Materials Testing Laboratory at Gross-Lichterfelde obtained: --

Mixture 1:3, resistance between 180 and 311 kil/cm².

Mixture 1:5, resistance between 119 and 226 kil/cm².

Mixture 1:8, resistance between 84 and 237 kil/cm².

The resistance of a cube increases with the age of the concrete (see p. 2 112). Experiments have shown, that the increase in resistance has often not entirely ended after 6 years. ²

Note 2. p. 204. According to Contributions of the French Commission for Reinforced Concrete, there are valid for reinforced concrete mixtures the following relations:--

Age	28	90	365 days.
Comparative resistance	100	150	250 per cent.

At the Munderking concrete Bridge, it was found by tests, ~~in~~ that the crushing resistance of the same concrete (1:2.5:5) had more than doubled after three years. The resistance of 202 kil/cm² after 7 days rose gradually to 570 kil/cm² after 9 years. (See Fig. 118).

Gary obtained (Heft 22) of German Commission) for a mortar 1:3 and different kinds of cement a crushing resistance of 122 (396) kil/cm² after 28 days. Thus the resistance of concrete 7 days old was about 70 p.c. of the resistance of concrete 28 days old.

As for what first concerns the compressible stresses in flexure, the Prussian Regulations prescribe the following.

"For structural members subject to flexure, the compressible stress in the concrete shall not exceed a sixth part of its crushing resistance." ¹

Note 1. p. 205. An upper limit for the allowable compressible stress in the concrete is not given by the Prussian Regulations, contrary to the regulations of other countries.

Here and in the following gravel or crushed stone concrete is generally considered. Pumice concrete (see p. 45) has less strength. Crushing experiments on an extended scale are still wanting; yet for mixing proportions of 1 cement + 2 quartz sand + 4 fine pumice gravel, one may assume about 120 kil/cm² as a limiting value for resistance. Allowable compressible stress ~~is~~ then = $\frac{120}{6} = 20$ kil/cm², according to Prussian Regulations.

Thus reference is to be made to the crushing resistance, and a sixfold security is to be taken. ² For mixtures 1:6 to 1:4, the crushing resistance after 28 days amounts to about 180 to 260 kil/cm², which value thus corresponds to $\frac{180}{6}$ to $\frac{260}{6} = 30$ to 44 kil/cm² for bending.

Note 2. p. 205. But the safety for the steel is only 2.0 to 2.5 fold, according to the usual mode of calculation.

By numerous experiments it is established, that the calculated stress in the concrete is often considerably greater than the resistance of a cube of the concrete, and that the values

only approximate in the vicinity of the breaking limit. ³ Likewise it has been proved by experiments, that about 2 (to 3) per cent reinforcement, an increasing load on the beam or slab does not produce fracture by preliminary crushing of the concrete, thus by exceeding the allowable stress in the concrete, but by exceeding the elastic limit of the steel, that thus in the structural member in flexure with the usual amount of reinforcement, the stresses in the steel generally indicate the degree of safety. One is more dependent on the material safety of the steel, and must then endeavor to make the fullest possible use of the steel cross section. ⁴

Note 3. p.205. According to Emperger for bodies under flexure the crushing resistance is about twice as great as the resistance of cubes, and the required maximum limit of $1/8$ the crushing resistance is equivalent to 11-fold safety. (Also see Note on p. 13). According to experiments of the German Commission (Heft 17), the bending resistance of a mixture 1:2.5:5 and of 1:4:8 is 18.7 p.c. of the resistance of a cube after 28 days. Schüle found the calculated crushing resistance of concrete in beams about 1.4 to 2.4 fold that in cubes. The usual calculation (neglecting the tensile resistance in the concrete, reinforcement of 2 p.c.), thus leads to an over estimate of the compressible stresses in concrete.

Note 4. p.205. Möller has proved, that it is not much to say for the safety of a structure, if the test of a cube remains somewhat below the required resistance, that a rectangular cross section must be reinforced up to 3 p.c., until finally the compressible stress in the concrete is determinative for the safety against breaking. Contribs. 1912. p.186. Also see Zeits. of Union of Architects and Engineers. 1912. p. 67.

p. 204 If one counts on 90 days instead of 28 days hardening of the concrete, then one may calculate with limiting values of 45 to 50 kil/cm² (see p. 204, 122). Attention is called to the French and the Hungarian Regulations (Table, p. 224), which are by far less hard than the Prussian Regulations. Likewise to the Wurtemberg Regulations (Tabel, p. 224), according to which the limiting stress values are dependent on the degree of expected vibrations. The Swiss Regulations even go to 70 kil/cm², while indeed the stress in steel is reduced to 600 kil/cm². (P. 222).²

Note 1. p.206. The French Regulations allow 28 per cent of the resistance of a cube after 90 days, which with a

resistance of 250 kil/cm^2 gives a stress value of 70 kil/cm^2 .

Note 2. p. 206. It thus appears advisable to take higher values as allowable compressible stresses in the concrete, than are ~~±~~ given by the Prussian Regulations. Attention should always be called to the fact, that a higher limiting ^{stress} ~~value~~ with an unchanged stress in the steel makes necessary a greater reinforcement. (See Section VIII). And that thereby the safety against formation of tension cracks in the tension zone of the concrete is reduced.

Dyckernoff & Widmann, Karlsruhe, loaded a reinforced concrete slab only 4.5 cm thick, continuous over 4 steel beams, 4 months old, with 1100 kil/m^2 live load (11 times the calculated $\pm$ load of 100 kil/m^2), without fracture. Spans 2.0 m, total loaded area = 23 m^2 , greatest deflection in the two end panels 8.5 cm and 5.5 cm. After complete removal of the load the deflection was reduced to 7.0 and 4.5 cm.

On the permissible resistance of supports to compression, ³ the Prussian Regulations prescribe, that the concrete shall not be stressed more than $1/10$ its crushing resistance. Accordingly the allowable values vary between $\frac{185}{10}$ and $\frac{260}{10} = 18$ and 26 kil/cm^2 for pure compression. The required factor of safety 10 is taken high. It would correspond more to the facts to fix a smaller factor of safety (for example 5 to 7), while at the same time for not reinforced concrete supports is prescribed a 10-fold safety. ⁴

Note 3. p. 206. On this see Sections XV, XVI, XVII.

Note 4. p. 206. Stresses of about 20 kil/cm^2 might thus be regarded as allowable for supports. According to the preceding principle for pure compression is permitted a limiting value of 35 kil/cm^2 . (Further see Contribs. 1908. ~~22~~ No. 3.

Decree of Prussian Ministry of Dec. 8. 1910:-- "The maximum compressible stress in lamped concrete for a quiet load must not exceed one fifth of its crushing resistance after 28 days. For supports and piers this stress is reduced with increasing ratio of height to least diameter, and at most is to be assumed $1/5$ of the resistance for ratio 1:1, $1/10$ for ratio 5:1, $1/20$ for ratio 10:1. Intermediate values are to be interpolated by straight lines. Tensile stresses in concrete are to be neglected in the calculation of the maximum compression in the angles." Also see Arm. B. 1911. p. 68.

According to Bach's experiments for obtaining the compressile resistance of concrete columns without reinforcement with different heights (Contribs. 1914, No. 5), the reduction of the stress need not go so far. On the ground of these experiments--considering the inaccuracies in the work -- one can count on about $1/8$ for the ratio 5:1, and about $1/12$ for the ratio 10:1.

But higher allowable stresses for slender columns than 30 to 35 kil/cm² are not to be recommended, since it is to be considered, that the load in general does not always act axially or uniformly distributed, as the calculation assumes.

For supports, that extend through several stories, if an increase of the allowable compressile stress were in place, and in consideration that only the upper thin supports might receive additional stresses in consequence of their rigid connection with the beams, and that for the supports of the lower story, a full loading of all story floors can scarcely come in question. (A reduction of the live loads according to page 163 must then certainly be omitted). Thus if one takes the difference from an exact calculation of the additional stresses, then one might begin with about 20 kil/cm² for the supports in the attic, and graduate the stresses downward to about 40 kil/cm² for the cellar piers. Mörsch even estimates 25 to 50 kil/cm², beginning from the top, as allowable values of resistance.

(See Section XV).

p. 208 b. Tensile Resistance of Concrete and of Reinforced Concrete.

Note 1. p. 208. Also see Section XIII.

It is generally required, that all flexure tensile stresses acting in reinforced concrete members shall be resisted only by the steel reinforcement, and thus any tensile resistance of the concrete is to be omitted in the calculations. Yet the Prussian Regulations prescribe:--

"For buildings or parts of structures, that are exposed to weather, wet, smoke, gases, and other injurious influences, it is further to be proved, that the occurrence of cracks in the concrete from the tensile stresses acting in the concrete will be prevented.

If in the designated cases the tensile resistance of the concrete is taken into account, then as allowable stress is to be taken two-thirds the resistance of the concrete to tension sh-

shown by experiments in tension. With lacking evidence of the tensile resistance, the tensile stress must not amount to more than one tenth of the resistance to compression."

For example, if a crushing resistance of 200 kil/cm² be obtained, then for a member in flexure would be:--

$$\sigma_{bd} = \frac{200}{6} = \text{about } 33 \text{ kil/cm}^2.$$

$$\sigma_{bz} = \frac{200}{10} = \text{about } 20 \text{ kil/cm}^2.$$

(According to the Regulations, this stress must correspond to a tensile resistance of at least $\frac{2}{3} \times 20 = 30 \text{ kil/cm}^2$).

For the usual cases in buildings, the participation of the tensile resistance of the concrete scarcely comes into consideration. It is much less than the crushing resistance (1/10 to 1/15 of that), and is greater for a rich, than for a lean mixture. According to experiments of German Commission (Heft 17), the tensile resistance of mixtures 1:2.5:5 and 1:4:6 after 28 days amounts to about 92 p.c. of the crushing resistance of a cube. As an allowable limiting value may be taken about 15 to 20 kil/cm².² Although the occurrence of tension cracks in concrete do not cause the destruction of the beam, yet the existence of a sufficient tensile resistance of the concrete is already desirable, since then the steel is relieved.

Note 2. p. 208. These limiting values have value only in calculations, since the tensile stress actually occurring with the assumption of equal changes of form for compression and tension (Russian Regulations) cannot amount to more than one half of those calculated. The Swiss Regulations fix 10 kil/cm² as the limiting value for tensile stresses at the edge of eccentrically compressed structural members. The limiting values for bending and eccentric compression given by the Austrian Regulations, according to the mixing proportions (22 to 25 kil/cm² are found in the Table on p. 224. In concrete members not reinforced 5 kil/cm² at most is to be taken as the allowable tensile stress.

On the ground of a later computed checking of Labes' rule¹ relating to the calculation of tensile stresses, on the basis of the results of flexure experiments, etc., the following have been established:--³

Note 1. p. 209. Labes, "How can the use of Reinforced Concrete in Railway Administration be substantially required?"

Cent. d. Bauw. 1906. No. 52.

Note 2. See B & E. 1907. p. 157.

Note 3. In the vicinity of the reinforcement the uninjured concrete can still sustain a tensile stress of 40 to 50 kil/cm². -- One can assume the tensile stress in flexure at about double the pure tensile resistance.

1. The calculated tensile resistance to flexure for tamped concrete work is smaller than that for reinforced concrete.

2. The tensile resistance to flexure increases with the amount of reinforcement, with the better arrangement of the cross section, and the roughness of the rods.

3. It likewise increases with the age of the concrete.

According to Tetmajer's experiments on test pieces of concrete and of reinforced concrete, 1.5 years old, the tensile resistance to flexure is considerably greater than pure tensile resistance, the steel reinforcement seems to substantially increase the tensile resistance to flexure. (Arm. B. 1911, p. 29, 31, 120).

Pure tensile resistance of concrete but seldom comes in question for the calculation of compound members, and can only be approximately obtained. ⁴

Note 4. p. 203. Note difficulties in test experiments; direction of force accurately in axis of test piece, avoiding bending moments. (Not allowable test values).

Each found by experiments tensile resistances of 17.35 and 25.10 kil/cm². As allowable limiting values one may take about 2 to 3 kil/cm² (with 10-fold safety).

~~b. Shearing Resistance of Concrete and Reinforced Concrete.~~

c. Shearing Resistance of Concrete and Reinforced Concrete. ⁵

Note 5. p. 209. Also see Section XII.

Proper consideration of shearing forces in reinforced concrete (by stirrups and beads in rods) is one of the most important points in the work of designing. According to the experiments of Mörsch, Bauschinger, etc., in all cases the shearing resistance of the concrete is about 1.2 to 1.5 times greater than its tensile resistance, and as in all other resistances, is larger for rich, than for lean mixtures. By experiments of the German Commission (Heft. 17), the resistance to shear for mixtures 1:2.5:5 and 1:4:8 is about 18.3 per cent of the resistance of a cube after 28 days. For a mixture 1:3, the shearing

resistance was found about 25 to 35 kil/cm². The official allowable value of 4.5 kil/cm² then corresponds to about 5 to 6 fold safety, assuming a rich mixture. It is more advantageous in each case to prescribe no fixed limiting value for all cases, but to make the value dependent on the required resistance of a cube.

The Austrian Regulations make the allowable limiting value dependent on the proportions of the mixture, the French and D-Danish Regulations on the allowable compressile stress, and the Württemberg Regulations on the intensity of shocks. (See Table, p. 224).

The shearing resistance of beams under flexure is generally less than the shearing resistance obtained by direct shear. Pure shear seldom occurs in reinforced concrete construction. On the ground of his experiments for obtaining the shearing resistance τ from the compressile and tensile resistances, Mörsch has established the equation $\tau = \sqrt{\sigma_c \cdot \sigma_t} \cdot 1$.

Note 1. p. 210. Mohr finds only one-half these values. Am. B. 1911. p. 249.

The Wayss & Freytag Co. (Professor Mörsch) undertook experiments on pure shear. By means of a Martens' press and according to Fig. 114, prisms of concrete without reinforcement, 40 cm long and 18 x 18 cm in cross section were tested. (See p. 123). It was found:--

For 1:3:., 14 p.c. water, 2 years old, about 65.9 kil/cm².

For 1:4, 14 p.c. water, 1.5 months old, about 37.1 kil/cm².

For reinforced concrete test pieces was obtained a resistance of about 35.5 kil/cm² at the age of 6 weeks. Mörsch finds no cooperation of concrete and steel in resisting pure shear. ¹

Note 1. p. 211. See Mörsch, Reinforced Concrete construction. In this work are sharply separated the two conceptions, "resistance to shove and to shear", indeed on account of the different appearances of the fracture in test pieces; yet such a distinction is generally unusual in practice.

Zipkes established, that the shearing resistance of concrete is materially increased by reinforcement. He found for test prisms 50 days old, ^{at} 1 : 3 with 14 p.c. water, 25 kil/cm² for concrete without reinforcement, and 57 kil/cm² for reinforced concrete. ²

Note 2. p. 211. Zipkes, Resistance to shove and shear of reinforced concrete. A

A criticism of this publication is contained in B & E, 1906. p. 289. (By Mörsch).

d. Bond Resistance between Concrete and Steel. ³

Note 3. p.211. Also see Section XII.

The actual existence of a bond resistance between concrete and steel most simply results from bending experiments with reinforced and non-reinforced concrete slabs; for the former slabs show a substantially greater supporting capacity, which is only to be explained, that the steel adheres firmly in the concrete, and contributes to the statical efficiency. ⁴ One ~~of~~ properly here has to do with a purely mechanical effect, which occurs when the concrete shrinks in hardening and clamps fast the reinforcement. Opposed to this view is another opinion, ~~as~~ according to which the silicates of the cement form an alloy with the steel on the surfaces in contact. ⁵ Whether the mechanical or chemical procedure takes the chief part in the bond is hard to say.

Note 4. p. 211. In other cases the insertion of the steel must coincide with a reduction of the cross section, thus with a loss of the supporting capacity.

Note 5. p.211. Rohland attributes the bond of the steel with the concrete to the colloid nature of the cement; by this is also compelled at the same time the shrinkage of the concrete. see Contribs. 1913. p. 103.

According to Preuss (Experiments on Bond between Steel and Concrete, Ann. B. 1910, p. 339), the bond between concrete and steel is to be referred to three causes, ~~which are:--~~ ~~1. Sticking in the~~ ~~XXXXXXXXXXXXXX.~~

1. The actual adhesion, i.e., sticking in the sense of "past-ing."

2. Resistance of friction, that opposes any slipping of the steel in the concrete, caused by contracting of concrete in setting, cramping the steel to a certain degree.

3. The wedging action of the sand grains, which in any slipping between the concrete and steel are loosened, and tend to prevent any further slipping.

The bond resistance increases with the diminishing effective depth of the concrete section and with the increasing percentage in reinforcing rods. As a rule round rods exhibit greater bond resistance than steel of other sections. The bond is the

greater, the richer the mixture, the slower setting the cement, and the older the concrete. Likewise the bond increases with the size of grains of sand and with the reduction of the water added. Practice has also shown, that effects of shocks, for example such as are unavoidable in factories and bridges, cannot injure the bond. ¹

Note 1. p. 212. Yet the Württemberg Regulations make the allowable limiting value of bond stress dependent on the degree of actually possible shocks. See Table on p. 224.

As for the allowable limiting values, it has been determined by experiments, that the bond in most cases is somewhat greater than the shearing resistance of the concrete; for example that many tests on rods pulled out of concrete, particles of concrete still firmly adhered. Therefore also in the Prussian Regulations the fixed limiting value of 4.5 kil/cm^2 (for allowable bond resistance), which must always be proved, is taken rather low. ²

Note 2. p. 212. But experiments have shown, that the bond has nothing to do with the resistance of the concrete to shear; see Section XII.

Men have proceeded from the point of view here, that the bond could no longer be effective, if the concrete were destroyed by excessive shearing stress. One must then assume higher values, if the stirrups and also anchoring and bending of all longitudinal rods are arranged. (See Section XIIU. The preceding principles assume 7.5 kil/cm^2 as the limiting value, corresponding to a maximum bond resistance of about 35 kil/cm^2 (thus a five-fold security). The French Regulations make the allowable bond stress dependent on the permissible compressible stress, ¹ and this again on the obtained resistance of a cube, which method is indeed most suitable, since it affords the advantage of a uniform safety, and creates the possibility of also increasing the bond by an improvement of the nature of the concrete. Engesser therefore proposes as the allowable value for bond stress (Arm. B. 1910, p. 71):--

0.1 of the allowable compressible stress with few longitudinal rods and a sufficient number of stirrups.

0.08 of the allowable compressible stress with many longitudinal rods and no stirrups.

Note 1. p. 213. See Table, p. 224. Each obtained the resistance to slip as $1/8$ to $1/10$ the resistance to crushing, and as

best results.

Note 2. p. 213. ^FAmperger estimates 4 kil/cm², with separate rods, cross bands and the like, 6 kil/cm². Mörsch and von Thullie prefer 7.5 kil/cm² for loaded beams under flexure. ^FFoerster estimates 7 kil/cm² as allowable bond stress under flexure and 5 kil/cm² for pure compression loading.

Different specialists, for example Probst, Saliger, Kleinlogel, Preuss and Schüle express the opinion, that the calculation of bond stresses may be omitted, so far as the other stresses do not exceed the allowable limiting values. The destruction of the beam results earlier by tension cracks in consequence of the stretch of the steel reinforcement. Likewise the Swiss Regulations allow bond stresses to be neglected, and also the Hamburg Regulations of 1913. ³

Note 3. p. 213. See Kleinlogel, "On the Nature and true magnitude of the Bond Stress between Steel and Concrete" Berlin. 1911; "The Possibility of omitting the Calculation of Bond Stresses" Contribs. 1911, p. 57, 66, as well as "Studies on the Question of the true Magnitude of Bond Resistance" Arm. B. 1910. p. 395; 1911, p. 287.

In construction a sufficient covering of the steel reinforcement is to be cared for. End joints in the steel are to be avoided, if possible (see p. 51). Some methods of construction exhibit the endeavor to make them independent of bond resistance, when special steel rods with rolled projections are employed (see p. 54) or flat bars with riveted angles (Miller plan).

Attention must always be called to the fact, that deformed bars of American patterns can only have the desired effect, if the rods are anchored in great masses of concrete, but that in the narrow ribs of T-beams is the contrary effect, when the steel exerts a splitting effect on the concrete at the bottom of the beam, so that a premature opportunity for the formation of cracks is afforded.

The diversity in the results of tests heretofore is to be referred in the first place to the varied nature and composition of the concrete mass, as well as to the different additions of water. Likewise the results are dependent on the different modes of making tests, whether the bond be obtained by pressing or pulling the steel rod out of the concrete mass (see Fig. 120), and whether the duration of the stress be short or long. In general, flexure experiments, even if difficult to make, c

limit of the steel. He found the magnitude of the resistance to slip to be 35 to 47 kil/cm². (Arm. B. 1911. p. 84).

Mörsch undertook pressure experiments with concrete cubes with 20 cm sides, 4 weeks old and rods 20 mm diameter (Fig. 92). With mixture 1:4 resulted the following mean values.

29.1 kil/cm² for 15 p.c. water.

31.2 kil/cm² for 12.5 p.c. water.

48.4 kil/cm² for 10 p.c. water.

The stress in the steel reached the maximum value of 2140 kil; thus the elastic limit (2600 to 3200 kil) was not exceeded. ² By adding 2 steel spirals around the main rods -- for the purpose of preventing a bursting of the concrete, the preceding values were increased to 54, 45.9 and 50.8 kil/cm².

Note 2. p. 218. In pulling experiments passing beyond the elastic limit, in consequence of the reduction of the section, a separation of the steel from the concrete occurs even before reaching the possible bond resistance. On Mörsch's experiments, see Mörsch, Reinforced Concrete Construction; also B & E, 1903, p. 273.

Von Emperger obtained by flexure experiments a bond resistance of 12 to 20 kil/cm² for round rods.

Also 20 to 30 kil/cm² for deformed bars.

All rods in flexure experiments, also those bent, were taken into account. ³

Note 3. p. 218. See "Researches in the domain of Reinforced Concrete." III and V.

Kleinlogel obtained for concrete beams 25 weeks old, mixed 1:3, a maximum value of 38.8 kil/cm². ⁴

Note 4. p. 218. See "Researches in the domain of Reinforced Concrete," I; also B & E. 1904. p. 257. Mörsch obtained in a similar way on a T-beam a bond resistance of 37.3 kil/cm², thus almost the same result as Kleinlogel. The above mentioned flexure values of Bach (17.0 to 22.7 kil/cm²) are substantially smaller, which indeed is to be referred to the fact, that there the rods were not anchored by bending the ends.

Bauschinger used test pieces 3 months old, mixed 1:3. For round rods 7 mm diameter, he found a bond resistance of 42 to 47 kil/cm².

Experiments of the Materials Testing Laboratory at Gross-Lichtenfelde showed that bulb-shapes had the best bond. Round

~~rods showed a resistance to slip of 31.6, 15.4, 8.2 kil/cm²~~

reciprocal value of the extension number n (see p. 228).

According to experiments of the German Commission (Heft 25) the assumption of $n = 15$ (corresponding more to the stage of fracture) appears entirely suitable, and also more favorable economically* than $n = 10$. The value 10 has as a result an increase of the calculated value of the stress in the concrete, but a reduction of the stress in the steel, since the zero line moves higher, x thus becomes smaller (see p. 241). The value 10 requires greater height n , and thus a greater weight of the concrete. ²

Note 2.p.228. See B & E. 1904. p. 107.

The Swiss Regulations make $n = 20$ or $= 10$, according to whether the steel lies in the tension or compression zone. This has as a result, that more steel is taken than for the assumption of $n = 15$. The stresses obtained with $n = 10$ will be smaller for the concrete and larger for the steel; Safety is thereby greater. Experiments of Schüle have shown, that steel in compression is not efficient to the degree formerly assumed, wherefore for steel in compression n is made $= 10$. *For Table 124, Appendix.*

By considering established by flexure tests, that fracture is preceded by a strong extension of the concrete, thus an elongation of the fibres in tension, 10 to 20 times as great as that for members not reinforced; our German experiments (Bach, Kleinlogel) have indeed not confirmed this, but it is well to assume, that the steel reinforcement (with a correct distribution of the rods) somewhat increases the capacity of concrete for extension, and at least delays the appearance of the first visible crack. ^{By} these experiments was established less capacity for extension of the concrete placed in air, in comparison to that placed in water; the concrete placed dry showed cracks in the tension zone under smaller stresses than beams placed wet.

are compressible forces. These are termed normal stresses and are designated by σ . They produce a change (λ) in the lengths of the different fibres, indeed shortened in the compression and lengthened in the tension zone. The changes in length referred to unity are termed extensions and are denoted by ϵ .

For example, if a tension fibre has the length l in cm in an unstressed condition of the slab, it will attain a length after bending $= l' = l + \lambda$ (tension) or $l - \lambda$ (compression). There occurs a parallel displacement of the end cross section by the distance λ . The extension of the fibre is then:—

$$\epsilon = \frac{\lambda}{l}$$

And the stress, if f = cross section of the fibre, is:—

$$\sigma = \frac{Z}{f}, \text{ or in general, } \sigma = \frac{P}{f} \text{ kil/cm}^2.$$

If one desires two slabs of the same cross sections and equal spans, one of which is made of steel and the other of concrete, the stresses in the corresponding fibres will be entirely different. Between the extension and the stress is the relation:— (Hooke's law):—

$$\epsilon = \alpha \sigma.$$

In which α is the coefficient of extension ($= \frac{\text{extension}}{\text{stress}}$), and which denotes for the different materials different extensions per unit of length for 1 kil/cm² of stress. The converse value of this coefficient is the modulus of elasticity E (see page

222) $\therefore$ thus $E = \frac{1}{\alpha} = \frac{\sigma}{\epsilon}$, consequently $\epsilon = \frac{\sigma}{E}$. 1 (1).

Note 1. p. 228. Therefore $\frac{\lambda}{l} = \frac{\sigma}{E}$. Thus a concrete beam 6 m long under a compressive stress of 40 kil/cm² will be shortened by $\lambda = \frac{40 \times 600}{142,000} = 0.158$ cm.

For some structural materials, particularly wrought iron and steel, according to formula (1), extensions and stresses have the same ratio at all cross sections (see p. 219) $\therefore$ thus α remains constant. For concrete this proportion does not exist. ² There prevails rather the relation (for compression):—

$$\epsilon = \frac{\sigma^m}{E}$$

p. 229.

In which formula for m , the so-called coefficient of change of form is to be inserted a value from experience (1.1 to 1.2) for the different mixtures. On the ground of this "exponent or law of change of form," one may exhibit the arrangement of all

~~concrete and steel reinforcement~~

extensions and stresses graphically by a diagram. In Fig. 126 $m o n$ is the extension curve, indeed a straight line by Navier's law. The two fibres f and f' are distant s and s' from the zero line, and receive the extensions ϵ and ϵ' .

$$\text{Then } \frac{\epsilon}{s} = \frac{\epsilon'}{s'}.$$

Note 2. p. 228. For concrete ϵ increases with the increasing stress, indeed more rapidly in tensile than compressive stress. (See p. 223).

The limitation of the stress diagram is the stress curve $p o r$. The stresses do not follow the law of elasticity: the measure of elasticity is also different for tension from that for compression.

The statistical conditions for reinforced concrete beams affected by flexure may be the following:-- (Fig. 127).

Case I. Beginning of the loading. At first, participation of concrete on the tension side. The steel is stressed to about $15 \times 20 = 300 \text{ kil/cm}^2$. (20 = tensile resistance of concrete).

Case II a. Increased loading. The proper tensile elasticity of the concrete is exhausted: the concrete extends in but slight measure (increased capacity for extension by the reinforcement, p. 226).

Case II b. Further increase of loading. The capacity of the concrete for extension is entirely exhausted. Cracks are formed, which extend almost to the zero line. ¹ The reinforcement must gradually receive all tensile stresses. The zero line is moved upwards.

Case III. Maximum allowable loading. Finally results fracture, indeed by exceeding the resistance to tension (or the elastic limit, see p. 213) of the rods, or the resistance to compression of the concrete. ²

Note 1. p. 230. One can consider the ruptured concrete to be cut out as in Fig. 128: $D = Z$.

Note 2. p. 230. In this case follows the destruction at middle of length of beam. As a rule, more quickly occurs the destruction in the vicinity of the support, indeed in consequence of shear cracks. (See p. 160 and Fig. 66 f).

The Prussian Regulations allow the use of Hooke's law of elasticity, thus the formula (1), according to which a ratio exists between extensions and stresses. The limiting curve of ϵ

the allowable compression is:—

$$\left\{ \frac{1}{4}, \left\{ \frac{1}{2}, \left\{ \frac{3}{4}, \left\{ 1, \left\{ 1 \frac{1}{4} \right. \right. \right. \right. \text{p.c. of concrete cross section in reinforcement.}$$

$$20, 30, 40, 45, 50 \text{ kil/cm}^2. \text{ (See Table p. 233).}$$

Calculation of the shear and bond stresses are only required for slabs in the rarest cases.

a. Formulae for Designing.

The application of the developed values for x , σ_b and σ_e requires a preliminary accurate statement of the thickness of the slab and of the sectional dimensions, as well as the number of the steel rods. But in designing a slab the previously unknown magnitudes are brought into the calculation as unknown quantities. The moment M of the external forces can be obtained with sufficient accuracy by a preliminary estimate of the slab load. Equations (5), (6) and (7) have the object of calculating the stresses σ for the given bending moment, given cross section and dimensions of reinforcement.

First let the moment M and the stresses σ_b and σ_e be given.

From formula (4) follows:— 1

Note 1. p.234. The distance to the zero line according to formula (3) represents a fraction of the effective depth, which is indeed dependent on the extreme stresses. Compare formula (5).

$$x = s h' \left(\text{if } s = \frac{n \sigma_b}{n \sigma_b + \sigma_e} \right). \quad (8).$$

If the value x of formula (8) be substituted in formula (6), then results:—

$$\sigma_b = \frac{M}{b s h'^2 \left(1 - \frac{s}{3} \right)} \quad (9).$$

If the value for x of formula (8) be substituted in formula (7), then results:—

$$\sigma_e = \frac{M}{f_e h' \left(1 - \frac{s}{3} \right)} \quad (10).$$

$$\text{By formula (2) is also found:— } f_e = \frac{\sigma_b - \frac{n}{2} \sigma_e}{\sigma_e} \quad (11).$$

For example, if $n = 15$, $\sigma_b = 40 \text{ kil/cm}^2$ and $\sigma_e = 1000 \text{ kil/cm}^2$. Then according to formula (8), $x = 0.375 h' = \frac{1}{3} h'$.

of f_e has been found, then a certain diameter of rod is chosen, and then the required number and also the distance between rods are found by the Table given in the Appendix.

If dimensions of a slab have been obtained, still in checking (perhaps in consequence of too low a value assumed for the dead load), too high stresses are found, these may be avoided in three ways.

1. Retaining the thickness of the slab and increasing f_e .
2. Retaining the value of f_e and increasing depth of slab.
3. Increasing both f_e and the thickness of slab.

In all these cases a reduction of the stresses in the concrete and the steel is produced, particularly by increasing the thickness n . Conversely, perhaps by a too favorable assumption of the dead load, neither of the two structural materials is stressed to the highest allowable value, then for economical reasons the dimensions of one material are reduced until it is fully utilized, thus attaining its highest stress.

The derivation of formulas for other conditions of stresses proceeds in the same manner as before: in the Tables given on p. 238, 239, these values are collected for different stresses σ_e and σ_b .. (For Tables see *Annex*, p. 209, 210.)

The Tables can also be used in the following manner:—

Let the given bending moment $M = 300$ m kil. The slab is to be 12 cm thick, 1 and rods with 1.2 cm diameter are to be employed. $\sigma_b = 40$ kil/cm², $\sqrt{M} = 28.3$, and $n' = 13 \rightarrow 1.5 = 10.5$ cm.

The given dimensions correspond to a definite value K for n' , that is found in the following manner.

$$n' = K \sqrt{M}. \quad K = \frac{n'}{\sqrt{M}} = \frac{10.5}{28.3} = 0.371.$$

The nearest value in the Table is for $K = 0.367$, indeed for $\sigma_e = 300$, $\sigma_b = 40$ kil/cm².

Then is required $f_e = 0.397 \sqrt{M} = 11.24$ cm².

Thus for 1 m width of slab, by Table of round rods (Appendix) must be used 13 rods for reinforcement. $f_e = 11.30$ cm².

In special cases interpolations can be made. For example, if $\sqrt{M} = 20$, then the process of calculation would be the following.

$$K = \frac{10.5}{20} = 0.525, \text{ thus } f_e = \text{about } 0.211 \times 20 = 4.22 \text{ cm}^2.$$

Finally and conversely with a given M and given cross section of steel, the required depth of slab may be obtained.

Let $M = 1000$ m kil and $f_e = \text{about } 11.30$ cm² (10 rods of 1.2

cm diameter.

$$\text{Then } f_e = K \sqrt{M}, \text{ thus } K = \frac{f_e}{\sqrt{M}} = \frac{11.30}{31.6} = 0.358.$$

The nearest values in the Table ($\sigma_b = 40 \text{ kil/cm}^2$) are $K = 0.337$ for $\frac{900}{40}$ and 0.397 for $\frac{900}{40}$. Then $h' = \text{about } 0.375 \sqrt{M} = 11.85 \text{ cm.}$

If a is assumed $= 2.15 \text{ cm}$, then $h = 14 \text{ cm}$.

For a given live load q (including flooring, plastering, etc.), to save the preliminary estimation of the dead load and the calculation of M , Henkel (Zeits. für Tiefbau, 1913, p. 106) has established for the usual spans of slabs the following designing formulas for $\sigma = 1000$ and 40 kil/cm^2 , $a = 1.5 \text{ cm}$.

$$h' = \frac{1^2}{4.4} + \frac{1}{7.2} \sqrt{q l^2 + 36}, \text{ for } M = \frac{q l^2}{8}.$$

$$h' = \frac{1^2}{5.5} + \frac{1}{8.1} \sqrt{q l^2 + 36}, \text{ for } M = \frac{q l^2}{10}.$$

$$h' = \frac{1^2}{8.2} + \frac{1}{9.9} \sqrt{q l^2 + 36}, \text{ for } M = \frac{q l^2}{15}. \quad 1$$

Note 1. p. 240. According to Example 1 a (p. 248), h' would = $\frac{2.10^2}{4.4} + \frac{2.10}{7.2} \sqrt{390 + 36} = 7.02 \text{ cm.}$

$$f_e = 0.75 \times 7.02 = 5.27 \text{ cm}^2.$$

Further Notes for the use of Table on p. 238.

With increasing σ_b , the values for f_e also diminish, but on the contrary, the depth h' increases. Thus one should not assume too weak a reinforcement, since otherwise too high stresses in the steel would occur. Likewise the resistance to compression lessens with stronger reinforcement. If one desires to save steel, then this is to be taken at the permissible limit of 1000 or 1200 kil/cm^2 ; but the concrete at only 80 to 30 kil/cm^2 . On the other hand to save concrete, or what is the same thing, to obtain a smaller thickness of the slab, then is taken for the concrete 40 to 40 kil/cm^2 as the maximum stress, σ but for the steel only a value of 750 or 800 kil/cm^2 . It is most economical for slabs to utilize f_e up to the permissible limit. It is also to be considered, that with values of σ_b becoming smaller, the thickness of the slab is reduced very little, of no importance in practice, but on the other hand, the required cross section of the steel rapidly becomes excessive, so that finally there is no economical advantage. (See Example 1). Resulting conclusion:-- one stresses the building material, which is to be economized, to the permissible limit, but the

The steel contained would then be (for $n = 15$):—

For σ_e and $\sigma_b = 1000$ and 40 kil/cm^2 ,	0.75 p.c.
For σ_e and $\sigma_b = 800$ and 52 kil/cm^2 ,	1.60 p.c.
For σ_e and $\sigma_b = 600$ and 60 kil/cm^2 ,	3.28 p.c.
For σ_e and $\sigma_b = 500$ and 70 kil/cm^2 ,	4.73 p.c.

The formula proposed by Sabille for the Swiss Regulations runs:

(see p. 225):—

$$\sigma_b = 40 + 0.05 (1200 - \sigma_e) = 70 \text{ kil/cm}^2 \text{ as a maximum.}$$

According to this formula then results the steel content ($n = 20$).

For σ_e and $\sigma_b = 1200$ and 40 kil/cm^2 ,	0.417 p.c.
For σ_e and $\sigma_b = 1000$ and 50 kil/cm^2 ,	0.834 p.c.
For σ_e and $\sigma_b = 800$ and 60 kil/cm^2 ,	1.610 p.c.
For σ_e and $\sigma_b = 600$ and 70 kil/cm^2 ,	3.126 p.c.

For full utilization of both structural materials ($\frac{1000}{40}$).

$x = 0.876 h'$. The zero line lies at $\frac{3}{8}$ of the effective depth:

Accordingly the lever arm between tensile and compressive forces $= (h' - \frac{x}{3}) = \frac{7}{8} h'$ of the effective depth. According to formula (7), then the required steel section for an assumed slab h cm thick with full utilization of both structural materials (M in m kil , h' in cm) ¹:

$$f_e = 0.115 \frac{M}{h'}, \left(\text{for } \frac{1000}{40} \right); f_e = 0.094 \frac{M}{h'}, \left(\text{for } \frac{1200}{40} \right) \quad (13).$$

Note 1.p.243. This approximate formula can be employed for depths of slabs from 6 to 12 cm, since with small pressures in the concrete no notable alterations occur.

On further desired formulas, etc., see among others, B & E, 1907, p.235; 1908, p.275, 282; 1909, p.92, 98, 101, 204, 268; Ann. B. 1910, p. 25; 1914, p. 60, 86; contribs. 1913, p.156, 165.

For accurate calculations then result stresses, that are still somewhat smaller than the inserted σ_e , so that a higher degree of safety exists.

$$\text{According to Koenen is approximately } f_e = \frac{M}{\sigma_e \times \frac{3}{4} d}. \quad (14).$$

For example, according to Example 1 A, p. 243:—

$$f_e = \frac{33,100}{1000 \times \frac{3}{4} \times 9} = 4.3 \text{ cm}^2 \text{ (instead of 4.85 cm}^2\text{)}.$$

7. The covering layer a must be so chosen, that between the bottom of the steel and the surface of the concrete exists an interval of 1 to 2 cm (Fig. 130). For example, for rods of 10 mm diameter, then one must take for a at least 1.5 cm. Using

p. 248

$$l = 2.11 \text{ m}; g = 0.11 \times 2400 = 264 \text{ kil m}^2.$$

$$Q = (800 + 90 + 264) \times 2.11 = 1380 \text{ kil.}$$

$$M = \frac{1380 \times 2.11}{8} = 364 \text{ m kil}; \sqrt{M} = 19.08.$$

For steel rods of 8 mm diameter, $h' = 9.6 \text{ cm}$, $K = \frac{9.6}{19.08} = 0.503$.

For stresses of 1000 and 29 kil/cm², $f_e = 0.221 \times 19.08 = 4.22 \text{ cm}^2 = 9 \text{ rods of 8 mm diameter. (} f_e = 4.53 \text{ cm}^2 \text{)}.$

Making the slab thicker has only made possible a very small reduction of the required section of the steel: the concrete furthermore is not fully utilized.

Final results of the investigation. (Fig. 133).

	σ_e	σ_b	Depth.	Reinforcement.
a	1000	37 kil/cm ²	9 cm	10 rods of 8 mm, $f_e = 5.03 \text{ cm}^2$.
b	800	39 kil/cm ²	8 cm	14 rods of 8 mm, $f_e = 7.04 \text{ cm}^2$.
c	1000	29 kil/cm ²	11 cm	9 rods of 8 cm, $f_e = 4.53 \text{ cm}^2$.

The most economical solution then occurs when steel and concrete are stressed as nearly as possible to the allowable limits. If one desires to reduce the stress σ_e in the steel, then the reinforcement of the slab is increased: if he wishes to reduce the stress σ_b in the concrete, then the thickness of the slab is increased.

Example 2. (Fig. 134).

The positive maximum moment of a floor panel amounts to 324 m kil. Besides, the occurrence of a negative moment M in the panel of 100 m kil is to be taken into account in the calculation (See p. 182, Figs. 87 to 90). Allowable limiting stresses are 1200 and 40 kil/cm². The compressive stress, sometimes coming into consideration is here neglected — to raise the degree of safety and to simplify the calculation. (Section IX).

a. Thickness of slab is determined by the positive panel moment. ($\sqrt{M} = 18.0$).

$$h' = 0.411 \times 18.0 = 7.4 \text{ cm}; h = 7.4 + 1.6 = 9.0 \text{ cm}.$$

$$f_e = 0.228 \times 18.0 = 4.1 \text{ cm}^2 = 9 \text{ rods of 8 mm, (} f_e = 4.53 \text{ cm}^2 \text{)}.$$

p. 249 b. Obtaining the reinforcement for the negative panel moment. ($\sqrt{M} = 10.0$).

Thickness of slab must remain 9 cm. Let a be here made = 3 cm, then $h' = 7 \text{ cm}$.

$$K = \frac{7.0}{10} = 0.70.$$

$$f_e = 0.127 \times 10 = 1.27 \text{ cm}^2 = 3 \text{ rods of 8 mm, (} 1.51 \text{ cm}^2 \text{)}.$$

Span for calculation = 3.4 m.

Dead weight = $3.4 \times 0.6 \times 0.38 \times 2400 = 1,860$ kil.

Weight of wall = $16,000$ kil.

Total load Q = $17,860$ kil.

Maximum moment = $\frac{17,860 \times 3.4}{8} = 7,520$ m kil.

p. 257 $\sqrt{M} = 87.2$; $\sqrt{b} = \sqrt{0.38} = 0.616$; $\sqrt{\frac{M}{b}} = 142$.

$h' = K \sqrt{\frac{M}{b}} = 56$ cm, thus $K = \frac{56}{142} = 0.394$.

For $\sigma = 1000$ and 39.5 kil/cm²,

$f_e = \text{about } 0.29 \sqrt{M b} = \text{about } 0.29 \times 87.2 \times 0.616 = 15.58$ cm².

= 5 rods of 20 mm, ($f_e = 15.9$ cm²).

Check:—

$$x = \frac{15 \times 15.7}{38} \left[\sqrt{1 + \frac{2 \times 38 \times 56}{15 \times 15.7}} - 1 \right] = 20.9 \text{ cm.}$$

$$\sigma_b = \frac{2 \times 759\,000}{38 \times 20.9 \times 49} = 39 \text{ kil/cm}^2; \quad \sigma_e = \frac{759\,000}{15.7 \times 49} = 968 \text{ kil/cm}^2.$$

2. Obtaining steel section for beam only 56 cm high.

It is evident that a considerably stronger reinforcement will be necessary: therefore the distance a should not be assumed too small, and the height h' not too great.

$$h' = 56 \div 5 = 51 \text{ cm.}$$

The reduction of the dead load is unimportant: thus again

$$\sqrt{\frac{M}{b}} = 142.$$

$$K = \frac{51}{142} = 0.359.$$

For $\sigma = 800$ and about 41 kil/cm²,

$f_e = \text{about } 0.400 \times 87.2 \times 0.616 = 21.5$ cm² = 7 rods of 20 mm.
($f_e = 22$ cm²).

To receive the shears are arranged stirrups according to Fig. 136. The ~~stress~~ ^{stress} $\sigma_b = \text{about } 41$ kil/cm², by the insertion of some compression rods (about 2 rods of 20 mm, that at the same time serve, (see Fig. 137) as aiding rods, can be reduced to the limit of 40 kil/cm².¹ Otherwise it is to be considered that the beam is calculated as freely supported, but actually a certain degree of fixing exists, and already by the adoption of $\frac{a}{l} = \frac{1}{9}$, the limiting stress of 40 kil/cm² is reached. In the worst case must one conform to a further increase of the reinforcement, which naturally results in a further reduction of stress. For example, if instead of the two rods in the compression zone, ~~additional rods are~~

two additional rods are placed in the tension zone, (thus $f_e =$
 9 rods of 20 mm = 28.26 cm²).

Then $x = 24.5$ cm, $\sigma_c = 39$ kil/cm² and $\sigma_s = 625$ kil/cm². 1

Note 1. p. 251. See p. 263.

Note 1. p. 252. A check by formula (14), p. 243.

$$f_e = \frac{759\ 000}{630 \times \frac{3}{4} \times 58} = 28.8 \text{ cm}$$

The required reduction of the height of the beam by only 4
 cm (from 60 to 56 cm) results in a considerable increase in
 in cost in consequence of the use of more steel.

Section IX. Calculation of doubly reinforced Concrete Slabs.

Doubly reinforced concrete slabs will first be employed with advantage, when one must consider the alternating occurrence of positive and negative moment, for example for division ^{walls of} ~~the~~ water reservoirs and elevators (see Fig. 141), for stairs, but also particularly for floors loaded with alternating traffic loads. (See Figs. 97, 99). In such cases, for a positive bending moment, the tensile stress affects the lower reinforcement; for negative bending moments the upper rods. Reinforcement for compression in general is then only suitable and economical, when with complete utilization of the compressible stress in the concrete, the tensile reinforcement itself cannot be fully stressed. The smaller the stress in the tension rods, the more advantageous the insertion of compression rods. As experiments ² have shown, these also allow a material increase of the breaking load, reduce the shortening in the compression zone, and also likewise the deflection of the beam. As a limiting value for the economy of a compression reinforcement may be assumed a tension reinforcement of about 0.5 to 0.6 p.c. of the total cross section. With a smaller tension reinforcement, the double reinforcement offers no special advantage.

Note 2.p.252. Experiments on reinforced concrete beams by the Stuttgart Materials Testing Laboratory (Contribs. on Research Work in the Domain of Engineering. Heft. 80, 91) have further shown, that the increase in the breaking load appears to be dependent upon the dimensions of the reinforcement and on its properties (trade iron, steel). The stirrups remain without apparent influence on the magnitude of the maximum load.

Doubled reinforcement will then also be employed with advantage, if the slabs or beams extending over several supports and with uniform depth have to receive negative moments at the supports. For the calculations here only the width b_1 of the beam comes into consideration. (Fig. 138). ¹

Note 1.p.253. With the requirement of a very limited structural height, a spiral reinforcement must be regarded as the most effective means for reducing the height of the beam. See Section XVII.

But the useful effect of the compression reinforcement for thick slabs is so unimportant, that the stresses in the doubly reinforced slab differ very little from the stresses in the

singly reinforced slab, it being assumed, that the reinforcement in the tension zones are similar to each other in size and location. For example, if one makes the total compression reinforcement f'_c equal to the tension reinforcement f_c , then the distance x to the zero line is reduced a little, and therefore the maximum stress σ_c in the concrete is somewhat reduced. As for the stresses in the steel, these will not be substantially different from those in the singly reinforced slab, while the existing maximum stress in the compressed rods eventually does not reach the maximum allowable value = 1000 kil/cm².

All this is true of slabs of considerable thickness. But if a limited depth is required, then a double reinforcement in such cases may be economically more advantageous. A single reinforcement then results frequently in overstressing the concrete; but by compression reinforcement the stress may again be brought to the allowable limit. As a disadvantage opposed to the economical advantage of a smaller thickness of the slab is always an important increased use of steel, and thus a higher cost.-- According to Fig. 139, the compression reinforcement f'_c increases gradually, beginning at zero, until it exceeds the value f_c of the tension reinforcement. Naturally the magnitude of the tension reinforcement also increases slowly:— (assumed constant M and constant maximum stresses).

The question of compression reinforcement also plays a great part in beam bridges with sunken railway (Fig. 140): here is first concerned a reduction of the depth of the girder. ¹ See Kersten, Beam Bridges. 3rd edition. Sect. B.

Note 1.p.254. Also see Note on p. 253.

Compression and tension rods in all cases may be connected by stirrups in order to prevent buckling of the compression rods. ² (See Fig. 144 b on p. 363, also Fig. 137). With slabs of sufficient thickness it is to be finally considered, whether it be not preferable, instead of compression rods to try a corresponding increase of the tension reinforcement. The zero line moves downward, and the stress σ_c in the steel thus diminishes considerably. ³ Since then experiments have shown that also the compressile resistance of the concrete in flexure increases with the percentage of the tension reinforcement, the increase of this tension reinforcement would be able to present a considerable increase of safety in a given case.

0.8	0.400	0.851	0.313
1.0	0.385	0.835	0.296
1.5	0.348	0.830	0.248
$x = 0.304 h' \quad x = 0.338 h' \quad x = 0.360 h'$			

$$\alpha = \frac{f'_e}{f_e} \quad f_e = w b h'$$

$\sigma_e =$ 1000 kil/cm ²	b in m, h' in cm.		
	$\sigma_b = 35$	$\sigma_b = 40$	$\sigma_b = 45$
0.0	$w = 0.602$	$w = 0.750$	$w = 0.905$
0.2	0.648	0.815	0.994
0.4	0.700	0.893	1.102
0.6	0.762	0.987	1.240
0.8	0.836	1.103	1.413
1.0	0.926	1.250	1.645
1.5	1.270	1.875	2.750
$\sigma_e =$ 1200 kil/cm ²	$x = 0.344 h'$	$x = 0.375 h'$	$x = 0.402 h'$
0.0	$w = 0.443$	$w = 0.555$	$w = 0.675$
0.2	0.471	0.595	0.730
0.4	0.502	0.641	0.794
0.6	0.533	0.694	0.871
0.8	0.579	0.757	0.965
1.0	0.626	0.833	1.030
1.5	0.789	1.110	1.543
$x = 0.304 h' \quad x = 0.333 h' \quad x = 0.360 h'$			

Determination of stresses in the steel reinforcement.

The mean tensile stress in the lower steel reinforcement f_e is found by the aid of the formula deduced on p. 256:—

$$\sigma_e = n \sigma_b \frac{h' - x}{x} \quad (18).$$

And the mean compressible stress in the upper steel reinforcement f'_e by the aid of the formula:—

$$\sigma'_e = n \sigma_b \frac{x - a'}{x} \quad (19).$$

As for what concerns the design of a doubly reinforced concrete slab, the relations found for a simple reinforcement are also true here. On p. 258, 259 is given a Table for obtaining the cross sections of doubly reinforced slabs and beams, made by Geyer, ~~Leipzig~~ Leipzig, which corresponds in form and arrangement to the Table on page 233 for single reinforcement. (See Arm. B. 1913. p. 81). The assumption is here made, that the

that the compression rods lie at the centre of gravity of the compression triangle, thus being at the distance $\frac{x}{3}$ from the top of the slab. ¹

Note 1. p. 2-9. That does not always occur. One makes a' as small as possible, about $\frac{1}{10}$ depth of beam, but never less than 1.5 cm.

But the case usually occurs, that the depth h is given, but single reinforcement no longer suffices, since then σ_b would be too high. Then one abandons single reinforcement and takes into consideration, that with this reinforcement the unequilibriumed moment is:—

$$M_2 = M_{\max} - \frac{b \cdot x}{2} \times \sigma_b \left(h' - \frac{x}{3} \right).$$

And this requires for the given stresses a special compression reinforcement f'_e and at the same time an increase of the tension reinforcement. Assumed is a complete utilization of the most distant stresses, given are h , b , a and a' (in m), σ_e and σ_b (in kil/cm^2), as well as M (in $\text{tonnes} \cdot \text{m}$).

Note 1. p. 280. On derivation of the formulas (20) to (22) see Arb. B. 1911. p. 53.

Then according to formula (8):—

$$x = \frac{n \sigma_b}{\sigma_e + n \sigma_b} \times h'. \quad \text{According to Löser, then;}$$

$$\text{Partial moment:—} \quad M_2 = M_{\max} - b \cdot x \cdot \sigma_b \left(h' - \frac{x}{3} \right). \quad (20).$$

$$\text{Compression reinforcement:—} \quad f'_e = M_2 \frac{1000(h' - x)}{\sigma_e(x - a')(h' - a')} \quad (31).$$

$$\text{Tension reinforcement:—} \quad f_e = \frac{5000 \cdot b \cdot x \cdot \sigma_b}{\sigma_b} + \frac{1000 M_2}{\sigma_e(h' - a')} \quad (32).$$

For the special case that $\sigma_e = 1000 \text{ kil/cm}^2$ and $\sigma_b = 40 \text{ k/cm}^2$,

$$f_e = \frac{M_{\max}}{h'(1 - m)} + 75 \cdot b \cdot h' \cdot \frac{1 - 3m}{3(1 - m)}$$

$$f'_e = \frac{5}{3 - 3m} (f_e - 75 \cdot b \cdot h'); \quad \text{if } m = \frac{n'}{h'}.$$

Another method for designing doubly reinforced slabs is the following: ² if the bending moment M and the cross section of the concrete are given, then the permissible bending moment M^0 is computed by the Table for slabs (p. 233) for the fixed maximum stresses, and also the corresponding cross section f_e^0 of the steel. (Thus without reference to a compression reinforcement). Then is:—

$$f_e = \frac{M}{M_o} f_e^o \quad (23).$$

$$f_e' = 3\left(\frac{M}{M_o} - 1\right) f_e^o \quad (24).$$

Further references are made to the papers published in the technical journals: for example, B and E. 1909. p.177, 225: 1910. p. 114, 138; 1912. p.72, 167 (Suenson), 201 (Landmann). Arm. B. 1911. p.10 (graphical solution). — Zeits. d. Aust. Ing and Arch Union. 1911. No. 13 (Lichtenstein's method), also see Arm. B. 1911. p.302.

Example 6. Fig. 143.

For a moment $M = 1600$ m kil the maximum stresses in a simply reinforced slab are obtained as:—

$$\sigma_b = 50 \text{ kil/cm}^2 \text{ and } \sigma_e = 911 \text{ kil/cm}^2.$$

Given: $n = 15$ cm, $a = a' = 3$ cm, $f_e = 16$ cm².

1. It will now be investigated, whether by the addition of a compressible reinforcement the stress σ_b may be reduced to the allowable maximum value of 40 kil/cm². The thickness of the slab must not be greater than 15 cm: further if the compressible reinforcement $f_e' = f_e = 16$ cm², then:—

$$x = -\frac{15 \times 32}{100} + \sqrt{\left(\frac{15 \times 32}{100}\right)^2 + \frac{30}{100}} (16 \times 2 + 16 \times 13) = \text{abt. } 5 \text{ cm.}$$

$$\sigma_e = \frac{36 \times 15(13 - 5)}{5} = 864 \text{ kil/cm}^2.$$

$$\sigma_e' = \frac{36 \times 15(5 - 2)}{5} = 324 \text{ kil/cm}^2.$$

$$\sigma_b = \frac{160000}{225(15 - 1.65) + 15 \times 16 \times 0.6 \times 11} = 36 \text{ kil/cm}^2.$$

2. What steel cross sections are to be chosen, if for the same thickness of the slab (15 cm) and the same moment (1600 m kil), the two structural materials are to be stressed to the maximum limits, $\sigma_b = 40$ kil/cm² and $\sigma_e = 1000$ kil/cm²?

According to Table of slabs, p. 258, $x = 0.375 \times 13 = 4.88$ cm. Compressive stress in concrete (according to p. 256), $D_2 =$

$$\frac{40 \times 4.88}{2} \times 100 = 9760 \text{ kil.}$$

Equation of moments (according to formula 16):—

$$160000 = 9760 \left(13 - \frac{4.88}{3}\right) + D_1 \times 11, \text{ thus } D_1 = 4457 \text{ kil.}$$

Condition of equilibrium: $Z = 9760 + 4457 = 14217 \text{ kil.}$

$$\text{Tensile force in steel: } Z = \sigma_e f_e. \quad f_e = \frac{14217}{1000} = 14.22 \text{ cm}^2.$$

Note 1. p.267. Or also $y = \frac{2}{3} \left[x + \frac{(x-d)^2}{2x-d} \right]$. If $x = d$, then $y = \frac{2}{3} d$.

To obtain the maximum tensile stress σ_e in the steel, as for slabs, equate the bending moment $M_{\max}$ to the moment of the internal forces, thus:--

$$M = Z(n' - x + y) = \sigma_e I_e (n' - x + y).$$

$$\sigma_e = \frac{M}{I_e (n' - x + y)}. \quad (29).$$

$$\sigma_b = \sigma_e \frac{x}{n(n' - x)}. \quad (30).$$

For the stress in the concrete as maximum σ_b according to formula (26) is as above.

Formulas (27) to (30) thus apply to the case when the compressive stresses in the rib are neglected. For deep and heavy beams, for example as usual in beam bridges, it is recommended to take into account the compression stress in the rib as in Fig. 151, since thereby is produced a reduction of the calculated stress value σ_b .¹

Note 1. See Kersten. Beam Bridges. 3rd edition. p. 105. An example is also calculated there.

For T-beams should not be assumed too high values for the limiting stress σ_b , in any case not over 30 to 35 kil/cm²: for according to Fig. 152, the fibres lying in the upper surface of the slab are differently stressed. Fibre a above the rib is stressed in tension in the direction 1 - 1 (bending of the slab), but in compression in the direction 2 - 2 (bending of the beam). But for the fibre b we have only compression in both directions (particularly in the direction 1 - 1: it may thus be also stressed in compression to a greater degree, and not be destroyed by tension, like fibre a.

The use of I-shapes, as evident from Fig. 153, makes stirrups and bending of rods unnecessary, when the upper flange of the shape is in the compression zone of the T-beam. Any special attention to shear and bond stresses is then not needed.¹

Note 1.p.269. Yet stiff reinforcements are but rarely employed, since they are not economical, particularly when they extend into the compression zone. Also for the use of such shapes in reinforced concrete construction, so far as justifiable consideration exists, in consequence of the different behavior of the structural materials

the concrete is not stressed higher than about 30 kil/cm², but σ_e is then fully utilized. With less height of rib the section of the steel in tension must be increased, and if necessary a compression reinforcement must also be provided. Such an increase of f_e (for constant M and constant dimensions of the rib), only reduces the stress σ_e in steel materially, but the compressible stress σ_b in concrete slightly. ¹

Note 1. p. 272. If for the same T-beam the bending moment and accordingly the steel section f_e be doubled, the stress σ_e in the steel is not materially changed; but the value σ_b is increased in a considerable degree. For a T-beam ($b = 250$ cm, $h = 60$ cm, $d = 10$ cm, $b_1 = 18$ cm) Mörsch finds:--

For $M = 1,430,000$ cm kil and $f_e = 30.8$ cm²; $\sigma_e = 378$ kil/cm²,
 $\sigma_b = 17.5$ kil/cm².

For $M = 2,860,000$ cm kil and $2 f_e = 61.6$ cm²; $\sigma_e = 383$ kil/cm²,
 $\sigma_b = 29.4$ cm².

For making a design it is of importance to know whether the zero line falls within the section of the ~~slab~~^{slab} or passes through the rib. If it falls within the slab (also see formula 89), then the Table on p. 233 is ~~also then applicable~~^{also then applicable} if the zero line falls but slightly above the bottom of the slab. ² According to the Table will be sufficiently accurate:--

$$n' = 0.50 \text{ (to } 0.60) \sqrt{\frac{xM}{\sigma_b}}, \text{ for } \sigma_b = 30 \text{ to } 23 \text{ kil/cm}^2. \quad (65).$$

Note 2. p. 233. With table on p. ~~233~~^{233 one} can design with sufficient accuracy in all cases. Take the maximum value for σ_e , but only a smaller value for σ_b . (Under 25 kil/cm²). With accurate checking, σ_e results somewhat smaller, on the contrary σ_b is somewhat greater.

For the same T-beam resulted:--

For $x = a = 12$ cm: $\sigma_b = 19.7$ kil/cm², $\sigma_e = 1170$ kil/cm².

For $x = 11.3$ cm ($x < a$): $\sigma_b = 19.8$ kil/cm², $\sigma_e = 1167$ kil/cm².

And with a different steel section:--

For $x = d = 12$ cm: $\sigma_b = 19.7$ kil/cm², $\sigma_e = 1084$ kil/cm².

For $x > d = 12.33$ cm: $\sigma_b = 19.7$ kil/cm², $\sigma_e = 1084$ kil/cm².

If the distance x to the zero line is markedly greater than the thickness d of the slab (Fig. 148), then is made the assumption in designing, that the compression force in the concrete acts at the middle of the slab, so that $(x - y) = \frac{d}{2}$ cm. ³

Note 3. p. 272. The highest ^{Position} ~~position~~ of the compression resul-

On a gallery structure in the City Hall in Dresden (Arm. B. 1910, p. 293) dead weight was to be saved. The breadth of the 23 cm reinforced floor slab was somewhat reduced between the beams toward the supports, as in Fig. 158.

At the fixed simple T-beams there continuous over the supports are known to occur negative moments, so that the tension zone with the required reinforcement there lies in the section of the slab, but the compression zone only of the width b_1 lies in the lower part of the rib. particularly appropriate here are double reinforcements, that make possible a smaller height of the rib. (Figs. 138, 175). Likewise reference is made to the use of spirally reinforced concrete (Fig. 159 b), which allows higher stresses in the arches. (Section XVII). For continuous bridge girders and other heavy structures, if necessary, one may vary the height of the compression slab as in Fig. 159e, the slab thus lying below at the place of the negative moments. The same purpose is attained lastly by special compression slabs as in Fig. 159 f, ^{or} 1 or by a corresponding widening of the rib as in Figs. 159 c and d. All these methods naturally increase the cost of construction, and can only serve as aids, only in cases where deep arches must be avoided for practical reasons.

Note 1. p. 247. Kersten. Beam Bridges. 3rd edition, 1912, 187. Arches at the supports ¹ as in Fig. 169 serve only reasons of beauty, since here usually but very small fixing moments arise in question. Widening the beam at the support as in Fig. 152 only creates a greater bearing surface. The masonry will then be stressed in a lesser degree; besides a better anchoring of the external walls by the floor is produced. A construction like Fig. 161 at the same time takes into account the existence of greater shears at the support.

Note 2. p. 249. Experiments of Koenig & Koenig Co. have shown, that a continuously reinforced concrete beam also acts as a continuous beam, and that the arrangement of strongly reinforced arches is of the greatest use. The economical advantages of the arches is shown by the following data, that afford conclusions for the main reinforcement of the ribs, also for the amount of steel employed without stirrups. (See Morsch, Reinforced Concrete Construction).

Beam I (with arches); breaking load = 34.4 tonnes, amount of steel = 75.3 kil.



For ribbed floors without hollow blocks, whose slabs may serve as the compression slab of an adjacent beam, it is to be considered, that the upper concrete slab, so far as according to Section 14, No. 6 a, it is regarded as a slab-shaped part of this beam, must have a thickness of at least 3 cm.

4. The shearing stresses are to be computed for the weakest part of the ribs without regard to the brick layers, and for narrow ribs in which the mortar can only be grouted, must not amount to more than 2.5 kil/cm^2 . Only for ribs of at least 6 cm width, in which an ordinary tamping of the concrete is possible, may a shearing stress of 4.5 kil/cm^2 be allowed for the concrete.

5. The separate ribs in general must be reinforced with but one rod: only when the rib at the height of the reinforcement has a width greater than 6 cm, may a corresponding greater number of rods be placed in the ribs.

In the determination of the distance a of the reinforcement from the bottom of the floor is to be considered the thickness of the existing bottom of the hollow block or of the plate set to make the underside without joints.

6. The bearing of the floor on masonry must be at least 18 cm, and is to be so constructed, that the pressure at the edge does not exceed the allowable limit.

7. The ribs under no circumstances are to be employed as stiffening beams, but for stiffening the building at determinate places, for example at the axes of piers, are rather to be arranged special stiffening girders. Likewise the rods are to be properly anchored into the masonry at the usual distances.

For ribbed floors without hollow blocks, if the ribs are longer than 5 m, for a safe distribution of the concentrated loads on several adjacent ribs, is to be arranged a transverse rib of sufficient stiffness, but not taken into account statically.

8. The distance on centres of the ribs shall not exceed 60 cm. If instead of hollow blocks several of Klein's blocks with grooved surfaces are employed, the distance on centres of ribs may be increased to 75 cm.

9. Floors with visible ribs in dwellings must receive a plastered ceiling beneath.

10. Floors extending between masonry on both sides must only be calculated with a moment of $\frac{p l^2}{8}$: only in special cases if the necessary fixing is shown, and the construction

It is permitted in the calculation of reinforced concrete beams, between which such floors are fixed, to reckon as in the compression zone of the beam, the concrete between the separate ribs in the area of the negative moments, and further, so far as it concerns ribbed floors of hollow blocks, also the upper wall of the inserted blocks.

For ribbed floors of hollow blocks the average thickness of the compression zone will be obtained by the formula:--

$$d = \frac{d_1 b_1 + d_2 b_2}{b_1 + b_2}. \text{ See Fig. 201. "}$$

c. Brick Floors.

For the proper construction and calculation of simply reinforced brick floors, the following regulations of the Prussian Minister of Public Works (Jan. 21, 1909) are of importance.

The regulations for the erection of structures of reinforced concrete for buildings of May 24, 1907, find proper application to plane floors of brick with steel reinforcement, so far as the static conditions, particularly the form and location of the steel rods, corresponding to the assumptions, on which are based the rules in Sections II and III.

The modulus of elasticity of brickwork may be assumed at the twenty-fifth part of that of steel ($n = 25$). The maximum compressible stress occurring in the slab of brickwork, assuming the use of cement mortar, shall not exceed 15 p.c. of the crushing resistance of the bricks shown by official proofs, and in no case may amount to more than 35 kil/cm².

A concrete upper layer for increasing the bearing strength, if less than 3 cm thick, shall not be considered in the calculation of the bearing capacity: for at least 30 cm but not over 5 cm thick, may the strength be computed by the preceding rules for brick floors with steel reinforcement, thus with $n = 25$. Yet if the zero line falls within the concrete layer, or if the latter has a thickness over 3 cm, then the floor is always to be calculated as a reinforced concrete floor, according to the regulations of May 24, 1907, thus with $n = 15$, when the bricks are to be regarded as only a filling in the tension zone. The mixing proportions of the concrete layer must not be leaner than 1 vol. cement to 3 vols. mixed gravel sand.

The shearing stress of the flooring bricks must not exceed 2.5 kil/cm².

~~Floors of slab form, that rest~~

$$P = \sigma_d \left[\frac{b x}{2} + \frac{n f_e}{x} e (2x - n) \right]. \quad (91).$$

$$P(x - v) = \sigma_d \frac{b x^2}{3} + f_e \sigma_e (x - a) - f'_e \sigma'_e (n - a - x).$$

For obtaining the distance x of the zero line there results from the last equation after removal of the σ -values the following relation:—

$$\frac{b}{3 n f_e} x^3 \mp \frac{b v}{n f_e} x^2 + (n \mp 2v)x = 2a^2 + n^2 - n(2a \pm v). \quad (92).$$

If the point of application of P lies within the section the upper signs are valid; conversely if it lies outside the section (for example with corbels), then since v becomes minus, the lower signs are to be taken. The solution of this equation of the third degree is most conveniently obtained by trial calculations.

If f_e and f'_e have different values ($f_e > f'_e$), and likewise a and a' ($a > a'$), then if $\frac{P}{f_e}$ be placed = α :—

$$\frac{b}{3 n f'_e} x^3 \mp \frac{b v}{2 n f'_e} x^2 + [a(a \mp v) + (n - a' \mp v)] = \alpha a(a \mp v) + (n - a')(n - a' \mp v). \quad (93).$$

If no compression reinforcement exists, then both α and $f'_e = 0$.

$$P = \sigma_d \left[\frac{b x}{2} + \frac{n f_e}{x} (x - a) - \frac{n f'_e}{x} (n - a' - x) \right]. \quad (94).$$

As for making a design, generally the following is to be stated:—

The highest allowable value for σ_d does not always produce the section best economically: it frequently is advisable to count on lower limiting stresses in the design. Both limiting values for σ_d and σ_e can never be attained: σ_e seldom is over 500 to 600 kil/cm².

As a rule the dimensions of the concrete section are given, thus b and h . If P falls outside the kern, then one may employ the following approximate calculation.

With the assumption of a homogeneous structural material (without reinforcement):

$$\sigma_d \zeta = \frac{P}{F} \pm \frac{M}{W} = \frac{P}{b n} \pm \frac{P e n}{b n^2} = \frac{P}{b n} \left[1 \pm \frac{e}{n} \right]. \quad (95).$$

Volume of tension prism (Fig. 213):

$$Z = \frac{1}{2} b \sigma_c y, \text{ if } y = \frac{\sigma_c}{\sigma_d + \sigma_c} n. \quad (96).$$

Since now the concrete cannot receive tensile stresses, the entire force Z must be transferred to the reinforcement, thus:

$$r_0^2 = \frac{2}{\sigma_e}$$

If σ_d still lies within the allowable limits ($\sigma_d \leq 40$), then is inserted for σ_e the maximum value, thus 1000 or 1200 kil/cm². But if one has obtained a higher value for σ_d ($\sigma_d > 40$), then is taken a lower value for σ_e , for example 900 or 800 kil/cm². In any case in checking, σ_e would become substantially lower, than was assumed in the design. But for σ_d one obtains an entirely usable value. Yet if σ_d be still too great, then would either n be increased, or a compression reinforcement be employed, which is better.

In regard to the calculation of excentrically stressed sections of supports, also see the literature concerned.

R. Wuczowski, "Simplest method of dimensioning the cross sections with excentrically applied compressile or tensile forces." B and E. 1911. p. 202, 247, 324. (With many calculated examples: simple method, also adapted for slabs and T-beam shaped sections of arches and frames).

K. Stock, "Dimensions of rectangular reinforced concrete sections under axial compression and stressed for bending." Arm. B. 1911. p. 333, 433. (With calculated examples and Tables of coefficients for $\sigma_e = 1000$ and 1200 kil/cm²).

M. Mayer, "Dimensioning of rectangular reinforced concrete sections for combined resistance." Contribs. 1910, p. 91: 1911, p. 7, 151, 151, 159.

F. Nemec, B and E. 1914. p. 43. (Simple approximate method).

Theini, "Calculating and Dimensioning of singly and doubly reinforced sections, stressed by compression and bending. Contribs. 1912. p. 94.

Contribs. 1911, p. 150. (Teute method), p. 157. (Landmann's method for the case of equal reinforcement), p. 139, (Marcus' method for T-beam section), 1913, p. 22, 30, 133.

B and E, 1910. p. 66 (Example). -- Arm. B. 1911, p. 227: 1913. p. 24 (Rossin method): 1914, p. 19 (Nielsen method). -- Clay Industries Journal, 1914. No 25. (Henkel method).

Zeits. f. Tiefbau. 1912, p. 91, 104 (Example): 1911, p. 31, 45, 65, 176, 192, 204: also B and E, 1913, p. 9, 353 (general investigation of piers).

On experiments with excentrically loaded columns, see "Research Work in the domain of reinforced concrete:" Heft 10 (Thuillie's experiments): also Thuillie, "Further experiments with excentri-

Section XVII. Spirally banded concrete.

Recently for concrete structures are frequently employed columns of spirally banded concrete, according to a method given for them by Considere, which has the advantage of particular slenderness. A single continuous spiral encloses the concrete nucleus, and produces an increase of 2 to 3 times in the supporting capacity in comparison with columns, that contain an equal amount of steel in the form of longitudinal rods. The spiral banding also allows greater stresses in the steel: it further prevents displacement of the bands by tamping. (Also see Fig. 51, p. 131). Ordinary bands indeed protect the longitudinal rods from buckling, but increase the resistance of the concrete in a very small degree. Further advantages are seen on p. 352 under 1 and 2.

The method of calculation of spiral bands is fixed by the Prussian Ministry of Public Works by the decree of Sept. 13, 1908.

Let F_D = total section of the concrete.

F_E = total section of vertical steel rods.

F'_E = section of an ideal vertical reinforcement, if the steel existing in the spiral band per unit of column be transformed into a longitudinal reinforcement of the same length with equal volume.

Then the ideal section of the column thus formed is:—

$$F_1 = F_D + 15 F_E + 30 F'_E. \quad (97).$$

The allowable P for the column is determined by:—

$$P = \sigma_D F_1. \quad (98).$$

In which σ_D denotes the allowable compressive stress in the concrete of the support, according to the existing regulations. The larger section F_1 resulting from the formula, however is only permitted so long as it does not exceed $2 F_D$. ($F_1 \leq 2 F_D$). See example 24, p. 359, taken from the official requirements.

In a later decree (Dec. 21, 1909), it is stated that the procedure in calculation in structures does not alone depend on Considere's treatment, but likewise on other spiral transverse reinforcement, having the same effect on the bearing capacity of the reinforced concrete.

The basal idea of such a spiral reinforcement consists in the increase of the strength by the addition of spirals, that prevent or delay the formation of slip surfaces.¹ The section of the column is mostly round or octagonal, also hexagonal, etc.

square only in the Abramamoff-Wagid mode of spiral banding. (See p. 356). The spirals as a rule are delivered at the building site ready for use. They are made in the workshop by winding at a turning lathe a straight rod around a hollow steel mandrel, that may be round or polygonal. ¹ After the form has been made 1 m high, the vertical rods are set, and in the prescribed manner, are connected by wiring ~~with~~ a spiral 1 m high. This is then tapped, a new spiral of 1 m is placed and the form is extended to correspond. To obtain a safe butt connection of the spirals, each laps for two or three turns.

Note 1.p.351. The destruction of a concrete prism by compression is generally by the formation of slip surfaces, on which the shearing resistance is overcome.

Note 1.p.352. On winding machines, see B & B, 1912, p.55.

The chief advantages of spiral banding are the following:--

1. Production of a small section of the column.

For example, if $P = 400$ tonnes, $\sigma_D = 50 \text{ kil/cm}^2$, then the support with the ordinary bands with 5 p.c. reinforcement of the least section is computed at:--

$$\sigma_D = \frac{F}{F_D + 15 \times 0.05 F_D} \quad (\text{See formula 70}), \text{ thus}$$

$$F_D = \frac{P}{50 \times 1.15} = \frac{400\,000}{57} = 7000 \text{ cm}^2 = 96 \times 96 \text{ cm}.$$

By spiral banding ($\sigma_D = 55 \text{ kil/cm}^2$) under similar conditions and according to the official regulations:--

$$F_D = \frac{P}{55 \times 2} = \frac{400\,000}{110} = 3636 \text{ cm}^2 = 76 \times 76 \text{ cm}.$$

2. Use of less steel for given dimensions of the support.

For example, for a cellar pier, $P = 300$ tonnes, $\sigma_D = 50 \text{ kil/cm}^2$, then with an ordinary banded support $35 \times 35 \text{ cm}$, the steel section (according to formula 76) is:--

$$F_e = \frac{1}{30} \left[\frac{P}{\sigma_D} - 1.075 F_D \right] = \frac{1}{30} \left[\frac{300\,000}{50} - 1.075 \times 35^2 \right] = 26 \text{ cm}^2.$$

$$\text{Longitudinal reinforcement at 0.5 pc. of } F_D = 36 \text{ cm}^2.$$

$$\text{By formula (76) } F_e = \frac{300\,000 - 30 \times 35^2}{15 \times 30} = 135 \text{ cm}^2.$$

$$\text{Adding about } 10 \text{ cm}^2 \text{ for ordinary bands} = \frac{10}{195 \text{ cm}^2}.$$

In the upper stories the difference is smaller.

From the previous experiments with spirally banded columns can be deduced the following principles. ¹

The compressible stress in concrete assumed for this calculation may be obtained in this manner, that the maximum extreme stress in the concrete, that according to the usual mode of calculation for reinforced concrete beams, that neglects the steel reinforcement in the compression zone, will be multiplied by the area of the enclosed cross section."

Thus it first shall be computed as if no compression rods existed: the compressible stress in the concrete assumed for further calculations is then placed equal to the product of the maximum stress at edge by the enclosed portion of the area, in order finally to calculate according to the preceding rules for spirally reinforced concrete. See Example 26. ⁵

Note 3. p.357. Also see Preuss, "Beams with spirally reinforced ~~concrete~~ compression zone." B & E, 1913.p.54.

To this decree is appended the following calculated Example. (Cent. d. Bauv. 1912. No. 17:—

Given, negative moment $M = -1\,000\,000$ cm kil. Tension reinforcement 6 rods of 22 mm diameter (22.31 cm^2): compression reinforcement 4 rods of 16 mm (8.04 cm^2), as well as a rectangular bent wire spiral of 7 mm diameter (0.33 cm^2), 25 cm high with 4 cm between turns.

$$x = \frac{15 \times 22.31}{30} \left[-1 + \sqrt{1 + \frac{2 \times 50 \times 51.9}{15 \times 22.31}} \right] = 24.3\text{ cm}$$

$$\sigma_b = \frac{2 \times 1\,000\,000}{30 \times 24.3 \left[51.9 - \frac{24.3}{3} \right]} = 61.61\text{ kil/cm}^2.$$

This compressible stress is assumed to be uniformly distributed over the enclosed sectional area of the beam.

$$P = 30 \times 25 \times 61.61 = 46\,200\text{ kil.}$$

$$F_b = 25 \times 30 = 750\text{ cm}^2.$$

$$F_s \text{ for 1 m length of beam} = \frac{100}{.04} \times 4 \times 0.25 \times 0.33 = 9.6\text{ cm}^2.$$

$$F_1 = 750 + 15 \times 8.04 + 30 \times 9.6 = 1156\text{ cm}^2.$$

$$\sigma_b = \frac{P}{F_1} = \frac{46\,200}{1156} = 40\text{ kil/cm}^2.$$

Example 24. ¹

Note 1. p.358. Taken from Russian Regulations, pub. No. 188.

A column of 40 cm diameter and $F_b = 1590\text{ cm}^2$ has 6 longitudinal rods of 2 cm diameter, or $F_e = 6 \times 3.14 = 18.84\text{ cm}^2$. The spiral reinforcement extending around the longitudinal rods for 40 cm diameter of the spiral rings has per m in height 20 steel rings of 1.4 cm diameter.

$$\sigma_D = \frac{P}{F_D + 15 F_e + 30 F_1} = \frac{250 \cdot 400}{5625 + 15 \times 33.81 + 30 \times 38.92} = 35 \text{ kil/cm}^2.$$

Example 26. (Fig. 231).

According to Fig. 231, the moment of beam at a support is - 25 000 m kil. $n = 70$ cm, $n' = 65$ cm, and $b_1 = 40$ cm.

Tensile reinforcement $f_e = 46.82 \text{ cm}^2$.

Then by formulas (5) to (7) are found:—

$x = 38.37$ cm, $\sigma_D = 69.2 \text{ kil/cm}^2$, and $\sigma_e = 990 \text{ kil/cm}^2$.

To reduce the stress in the Concrete to the allowable limit value of $\sigma_D = 40 \text{ kil/cm}^2$, the overstressed part of the compression zone is strengthened by spiral reinforcement. According to the Fig. the height of this portion is:—

$$s' = \frac{\sigma_D - 40}{\sigma_D} \times x = \frac{69.2 - 40}{69.2} \times 38.37 = \text{abt } 15 \text{ cm.}$$

Assuming a uniform distribution of the edge stress over the enclosed part of the section, the compressile force in this portion is:— $P = s' b_1 \sigma_D = 15 \times 40 \times 69.2 = 42 \text{ 000 kil.}$

$$F_1 = \frac{P}{40} = \frac{42 \text{ 000}}{40} = 1050 \text{ cm}^2.$$

$$F_D = s' b_1 = 15 \times 40 = 600 \text{ cm}^2 \left(> \frac{1}{2} F_1 \right).$$

Longitudinal rods: $F_e = 0.5 \text{ p.c. of } F_D = 0.005 \times 600 = 3 \text{ cm}^2$.
= 4 rods of 10 mm, (3.14 cm²).

$$\text{Spiral reinforcement: } F'_S = \frac{F_1 - F_D - 15 F_e}{30} = \frac{1050 - 600 - 15}{30} \times 3.14 = 13.43 \text{ cm}^2.$$

Chosen are rods of 8 mm with $f'_e = 0.50 \text{ cm}^2$: no. of turns per lineal m, : $w = 12.5$ turns.

$$F'_S = 2(2 s' + 2 b) f'_e w = 2(2 \times 15 + 2 \times 40) \times 0.5 \times 12.5 = 13.75 \text{ cm}^2.$$

$$\sigma_D = \frac{P}{F_D + 15 F_e + 30 F'_S} = \frac{42 \text{ 000}}{600 + 47.1 + 413} = 39.7 \text{ kil/cm}^2.$$

The spiral reinforcement is according to the system of Abramoff-Magid. The shears are obtained in the usual manner and are so received.

Spiral Reinforcement of Cast Iron, Emperger System. ¹

For spirally reinforced concrete, the spirals increase the resistance of the internal concrete to compression: on the contrary for spirally reinforced cast iron they indeed safely retain the concret covering of the cast iron until fracture, thus aiding in preventing this from separating. Aside from protection from fire and rust, the concrete combines the separate cast portions together, and with its steel spiral banding

portions together, and with its steel spiral banding makes the brittle cast iron elastic and flexible, increases the section of the support, and ensures a statical cooperation of the entire cross section up to breaking. In the experiments appeared at fracture an entirely elastic behavior: the changes in form produced by the loading returned again to zero. With a considerable increase of the bearing capacity, one has the advantage of a small section of the member, thus a great slenderness of the compression member (columns). Notable is the following comparison of different round columns, on the basis of a total $\pm$ load of 265 tonnes.

Note 1.p.361. Further see Emperger, "Recent arched bridges of spirally banded cast iron," as well as B & E, 1912, p 57, 116: 1913, p.34, 137, 365; "The Foundry," 1914, Heft 5, 6 & ("Experiments on allowable loads on columns of spirally banded cast iron").

	Banded Cast Iron	Banded Concrete	Reinforced concrete
Area of section	0.13 m ²	0.50 m ²	0.70 m ²
Concrete per lin. m.	0.10 m ³	0.50 m ³	0.70 m ³
Steel in rods and spirals			
per m lineal	18 kil	172 kil	123 kil
Cast Iron, per m lineal.	130 kil	///---	-----

OMISSION.

On page 208 of this translation (page 242 of the original) paragraph 5 was omitted., and is given here.

5. The Table relates to plates with $b = 1.0$ m width for calculation. For designing the moment M is to be inserted in M kil for obtaining the stress in cm kil. (See Example 1. p. 246). The table may likewise be employed for T-beams indeed, if the zero line falls in the cross section of the slab. (See Section X).

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 Ratio of Prussian General Württemberg Austrian Regulations
 Moduli of Elasticity: 1907 German 1912 1 1911.
 Elasticity: 1904
 Concrete Union
 1904

$n =$

Ratio of
Moduli of
Elasticity:

$n =$ 15 15 15 15

Flexure.

Compressile $1/6$ the 40 to 40 (30) 42 with 470 k cement
 stress σ_c resistance 50 (likewise for 37 with 350 k cem.
 in to excentric 32 with 280 k cem.
 kil/cm² crushing compression) per m³.
 (same for excentric
 compression).

Tensile stress σ_t in 2/3 tensile --- For excentric 25 with 470 k cem.
 resistance or 1/10 the compression at 24 with 350 k cem.
 kil/cm² crushing resistance edge 6 (4). 22 with 280 k cem.
 (Same for excentric
 compression).

Shearing stress τ 4.5 k/cm² 4.5 4 (3) Values 4.5 with 470 k cem.
 in kil/cm² only 1/5 res- 4.0 with 350 k cem.
 distance to in 3.5 with 280 k cem.
 shearing. per m³

Bond stress $\tau_1 = \tau_0$ 7.5 4 (3) brackets 5.5 with 470 k cem.
 τ_1 in kil/ for 5.0 with 350 k cem.
 cm² heavy 4.5 with 280 k cem.
 snooks. per m³.

Compressile $1/10$ the 35.0 38 (27) 28 with 470 k cem.
 stress in crushing 35 with 350 k cem.
 supports. resistance 22 with 280 k cem.
 σ_c in
 kil/cm²

Tensile and 1000 and 1000 1000 (750a 1000
 compressile 1200 and more (least ten-
 stress in (Decree of side resist-
 mild steel Apr. 22 ance = 3800
 σ_t in 1913. kil/cm².
 kil/cm² see p. 319

Notes: Required least crushing resistance = 170
 Required least crushing resistance = 150 kil/cm² for 470 kil cem.
 Required least crushing resistance = 130 kil/cm² for 350
 Required least crushing resistance = 130 kil/cm² for 280 kil cem.
 Required least crushing resistance = 130 kil/cm² for 1 m³ tamped con-
 crete (after 6 weeks).

Note 1. p. 224. Considering the variations of temperature the allowable stresses will be exceeded about $1/3$, and about $1/2$, if the shrinkage be considered at the same time (fall up to 15°C).
 Note 2. p. 224. The lower limit is to be taken, if the diam. of the longitudinal rods = $1/10$ least dimension of the structural member, and the dist. bet. cross reinforcement corresponds to this dimen. The upper limit is taken if the diam. of the rods = only $1/20$ of least sectional dimension, and dist. bet cross reinforcement amounts to $1/3$ this dimension.

Allowable Stresses in Reinforced Concrete.

French Regulations. 1906. English Regulations. 1907. Swiss Regulations. 1909. Hungarian Regulations. 1909. (Swiss Commission on Reinforced Concrete). (Hung. Eng. & Arch. Union.)

8 to 15, according to diameter of longitudinal rods and dist. bet. transv. Reinf't. 15 a. Bending; for steel in tension = 20. For steel in compression = 10. b. Central or eccentric compression = 10 15

38 p.c. of cube resistance after 90 days. 42, or 1/4 the resistance to crushing. a. Compression in ribs and beam ribs near the supports = 40. b. Compression in beams of rect. Section finished cement and beam ribs near the supports = 40. At least 300 kil cement for 1 m³ maximum not over (40 + 0.05(1200 - 40)) 45

--- a. Bending. --- b. Eccentric compression = 10 at edge.

1/10 allowable compressible stress. 4.2 4.0 5.0

1/10 allowable compressible stress. 7.0 --- 6.0

38 p.c. cube resistance, 60 p.c. for spiral reinforcement. 35, or 1/5 crushing resistance. a. Axial compression = 35, neglecting bending stress. b. Eccentric compression (in the gravity axis) = 35, if this is considered, 45 with axial compression, 50, if all acting influences are considered. 38, neglecting bending stress, 45 if this is considered, 45 with axial compression, 50, if all acting influences are considered.

1/2 elastic limit (1/10 of this stress for shocks). 1050, or 1/2 elastic limit 1200 1200

For members subjected to alternating stresses and shocks, the allowable stresses reduced by 25 per cent. Least crushing resistance required = 170 kil/cm² (after 28 days)

Least resistance to crushing = 140 kil/cm² (after 6 weeks).

Note 3.p.225. σ_s = maximum safe stress in steel with exact consideration of effects of temperature and shrinkage, and are allowed σ_s = 70 kil/cm² and σ_s = 1500 kil/cm². The limit stress of 70 kil/cm² can only be employed when the stress in steel is reduced to 600 kil/cm², if the effects of temperature and shrinkage are neglected. Note 4.p.225. The allowable compressible stresses in supports are only valid, if the free length ≤ 15 times least dimension of cross section.

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TEIL II

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C. KERSTEN
Berlin 1915

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Translated by N. Clifford Ricker.



fixing and stiffening the form may also be employed hooks or bolts, that fit into corresponding holes. To equalize at the ends are arranged guides with or without sheet facing. The shape of these grounds may be as in Fig. 21 e or f.

In Fig. 22 is represented the section of an Ast floor (Ed. Ast & Co, Vienna), the ribs are about 50 cm apart. The illustration shows the addition of a plane ceiling.

Best known is the Koenen's plane ceiling. This is a slab having ribs or hollow spaces, that is connected with a plane ceiling extending continuously beneath the ribs. In consequence of the hollows the plane floor is very light, and still protects from sound and heat. It is also particularly suitable for the addition of circuits of all kinds, as well as for ventilating ducts to the stoves.

Transverse and longitudinal sections in Figs. 23, 24, show that under the ribs freely supported wooden timbers or strips are arranged, that may have different heights. The strips make possible both an easy construction of the ribbed slab as well as a convenient fastening of the plane ceiling beneath. In the ribs of the bearing slab are bedded steel rods of 1 cm diameter, naturally as low as possible. Thickness of compression slab 5 to 10 cm. With the small distances of 25 cm on centres of ribs, the plane ceiling is formed in the simplest way by nailed reeds and plastering, or fastened by zincked wires, (without centering), by plaster board, clay slabs, wire netting and plaster, and the like, or an ornamental ceiling of stucco or of wood may be directly fastened. The plane ceiling extends continuously beneath the ribs, so that a fracture of it along the ribs or an appearance of the latter cannot occur. For the purpose of tamping, the wooden beams support sheet iron moulds about 1 m long, that can again be removed after tamping.

Greatest distance between girders is 3.5 m. For heavy loads the effective height ($h - a$) is increased by extending the concrete ribs down to the ceiling beneath. The wooden beams placed correspondingly lower are removed with the sheet metal forms after tamping. The ceiling beneath is fastened to the concrete by embedded supports of zincked wires.

3. Slabs with hollow blocks and plane under surface.

Floors of reinforced concrete with hollow blocks are especially suitable for houses and commercial buildings; they possess two advantages over solid floors

Further, especially for mansard roofs, a slate covering may be employed. This can either be nailed on wooden sheathing fastened to wooden strips set in the concrete, be fastened to iron bars, or directly on the concrete (pumice concrete). Metal laid directly on the concrete without an intervening covering is unsuited as a covering material, since it conducts heat too well, thus producing injurious stresses in the concrete, made apparent by the appearance of cracks. In all cases is further advisable a copper covering on domes, churches and tombs. Sheet zinc is not to be employed without wooden sheathing or a 1 layer of pasteboard, since it is rapidly corroded by rich concrete. Finally also tile coverings come in consideration, for reinforced concrete roofs, indeed for an inclination of the roof equal or greater than 38° .

Frequently exists a need for a special insulation. A very good insulating material is a layer of cork 3 to 4 cm thick, laid directly on the concrete and covered watertight with tiles or roofing felt. ¹ Likewise a protection from heat is also as a good material, a 6 to 8 cm layer of cinder or pumice concrete, than can be nailed into during the first month, for preventing the dripping of the condensed water during the cold of winter, caused by warming the attic. For the purpose of avoiding all formation of drops is then recommended the making of holes directly below the surface of the roof in order to make renewal of air possible. ²

Note 1.p.100. A separate cork ceiling suspended from the frame of the roof is dear and has little effect, though creating an insulating hollow space.

Note 2.p.100. On the use of pumice concrete ceilings, see p. 13; on suspended Rabitz or Duro ceilings, see p. 103.

The arrangements for removing water from concrete roofs present nothing new. The gutters are also sometimes constructed of reinforced concrete (Fig. 275). As shown in Figs. 210, 211, the suspended gutters may be omitted, if by a corresponding execution of the concrete, the water is conducted to the different leaders.

2. Roof surface on steel framework.

Figs. 212 to 215 show different possible ways of constructing reinforced concrete surfaces of steel roof trusses. ¹ Such surfaces are nearly 5 cm thick and are constructed on wooden cen-

If the steel trusses are to be protected against fire, then a light Rabitz ceiling may be suspended from the lower chords, which as seen from the interior produces the appearance of an arched roof. (Fig. 221).

3. Beam roofs.

Beam roofs in reinforced concrete are generally to be designed according to the same rules as beam floors described on p. 32 (with smaller live loads). This is especially true of flat roofs, that very frequently come in consideration for industrial plans.

For the calculation as a rule the following live loads are determinative.

1. For an inclination of 0° to 15° the roof surface is treated as a horizontal slab and calculated for a live load of 250 to 300 kil/m^2 .

2. For an inclination of 15° to 60° is assumed a vertical load of 150 to 200 kil/m^2 for snow load and wind pressure.

3. For an inclination over 60° , one only needs to take the wind pressure of 100 kil/m^2 into the calculation, since the lying of the snow on the roof is excluded.

The danger, that on account of temperature changes cracks may form in the concrete, naturally exists in roof surfaces in a higher degree than in intermediate floors. If no insulating material (p. 90) is used to a sufficient extent, then expansion joints are to be placed at distances of 20 to 30 m, indeed lengthwise as well as crosswise the axis of the structure. The joints are closed either by filling with an elastic material, or by suitable arrangement in the covering of the roof. The division joints can only extend through the slab (Fig. 222). Better is a continuous division through beams and supports, (Fig. 222 a), or only through a beam (Fig. 222 b). Slip joints as in Fig. 222 e are generally less advisable. Foundations are not to be divided so. ¹ For lighted sheds, the skylights can be separated according to Fig. 291.

Note 1.p.104. Also see what is said on p. 88.

The flat roof of a high reservoir building near Metz, represented in Fig. 223, is covered by wood-cement. Distance between girders is 5.6 m, clear span is 16.8 m. Even the roof house is constructed in reinforced concrete.

Fig. 224 shows a roof of a moving picture hall with vaulted

With this sort of piles one is bound to no fixed time for delivery; he can proceed at once with the foundation after the award of the contract. The making of the necessary hole in the earth is done by boring (Strauss system) or by ramming (Simplex system). Objectionable is the necessity for an especially careful supervision, in particular concerning the existence of ground water, as well as the fact, that mechanical and chemical effects of the ground on the still fresh concrete -- so far as no metal case remains in the ground -- may be unfavorable. As a rule is also desirable here a reinforcement, indeed in view of a better connection with the footing (Fig. 365).

The Strauss process consists in this, that in the ground to be strengthened is bored a hole and at the same time a feed pipe is forced down (without pile driver). This bored hole is then tamped with concrete, drawing up the pipe to correspond. Thus by heavy tamping the concrete is forced into the cavities of the ground and forms a cylinder, that may be in different diameters according to its strength, and which essentially increases the bearing capacity of the ground in consequence of a uniform compressing of the loose and yielding layers of the soil. The round bulges also substantially increase the freedom, and by the increased labor a pile results, that possesses a most uniformly high resistance for its entire length.

The number of holes bored may be increased to correspond to the area of the building site. Work can be carried on at once, for a previous tamping in special forms and hardening before driving does not come in consideration for the Strauss piles. The hole is produced by boring, thus without any shocks. The appearance of the ground water presents no difficulties; but on the other hand any ground however firm can also be bored through if necessary. Then is added the advantage that for each separate pile one obtains valuable data on the exact nature of the ground itself, and further by the settlement of the mass per unit length of concrete tamped. The heads of the piles are joined together by a reinforced slab, whose reinforcement extends down into the upper parts of the piles, forming the proper foundation mass for the masonry placed thereon.

Fig. 364 gives a comparison of the originally intended foundations (hatched) of a locomotive shed with cleaning pit in S. Gall (sand filling with tamped concrete footings) with the

executed Strauss pile foundations. Fig. 365 shows the later insertion of a column by means of a Strauss pile foundation. Unquestionably in such cases bored piles offer special advantages in consequence of the absence of all driving. ¹

Note 1.p.151. Both Figs. 364, 365 are taken from the author's design for "Strauss pile foundations in Switzerland." Swiss B. Bauz. 1912. Vol. 59.

Further see Gehler, Strauss' patent concrete pile. Wm. Ernst & Son. Berlin.

For the Simplex pile foundation, a steel tube of 40 cm diameter with a cast iron point is driven into the ground, and as the tube is pulled up, the concrete is deposited in layers and tamped -- just as for Strauss piles. Instead of the cast iron point, that remains in the ground, the steel tube may be closed by a so-called alligator point, which opens when pulled up, and the concrete is tamped in.

Piles of steel plate (Stern, Raymond, Jansen systems, etc.) as a rule have conical form. The steel tubes remain in the ground; they are driven, either with or without previously driving a pile. ¹

Note 1.p.152. For driven steel tubes care should be particularly taken, that the point of the tube is sufficiently strong to penetrate the ground, and to prevent the entrance of water.

Crane Rails on Concrete Piles.

An example of the foundation of a portable bank crane is represented in Fig. 366. To obtain sufficient stiffness sidewise transverse frames are arranged on two piles at certain distances as in the illustration.

Fig. 367 shows a crane railway for the dock storehouse of the Aken Co. All piles were furnished with arrangements for sinking with a nose. Here it was found better to force the water through the interior of the pile than through a tube attached to the exterior of the pile. The pile frames stand at distances of 4 m apart. Further see B & E. 1911. p. 217.

On landing piers, laundry and loading sheds on reinforced concrete piles, see Figs. 422, 423, 524, 525, as well as Kersten, Bridges of Reinforced Concrete. Part I. p.205.

Sunken Wells.

Sunken wells of reinforced concrete are masonry wells capable of resistance to external forces and chiefly require little choice.

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choice. To them is generally given a round cross section, and the walls are made on the Monier system. They are sunk by extra loading, filled with concrete, and then serve for founding buildings and bridge piers.

The diameter of the well is to be made sufficiently great, so that only the filling of the well and not the circle will be loaded.

A very remarkable construction with wells, carried out by the designers Terner & Chopard, Engineering Office in Zürich, is represented in Fig. 368. A residence lay over a railway tunnel, but whose walls must not be loaded more than was already the case. The difference in height from bottom of tunnel to cellar floor amounted to 14.0 m. The ground was a good moraine in solid layers; the angle of repose was therefore assumed at 45° . For transferring the weight of the part of the building next the tunnel to the required depth served two wells of 3 m external diameter, furnished at bottoms with a strong steel circle. The entire weight of the building was transferred by reinforced concrete girders, partly to the wells, partly to the firm ground. ¹

Note 1. p. 153. Further see Concrete and Reinforced Concrete Structures. 1911. Bern. p. 1.

B. Cellars, Watertight Cellars.

The arrangement of dry cellar rooms is often quite difficult on account of unfavorable conditions of the ground water, particularly when the bottom of the cellar lies below the water level in the ground. A later keeping out of the water requires considerable expenditure and seldom corresponds to expectations.

One avoids all direct contact of the concrete with the water, if the latter contains gypsum. Insulating materials are coatings of tar, asphalt slabs, cement mortar consisting of 1 cement + 2 to 2.5 sharp sand + 0.1 milk of lime (applied in 2 or 3 coats, and after setting, covered by a thin layer of neat cement mortar) tarred roofing board (strips lap 10 to 15 cm over each other and are cemented with hot tar), Siebel's Asphalt-lead insulating sheets (2 to 4 mm thick of tarred board give the lead sufficient protection from chemical, physical and mechanical influences) and many others. ¹

Note 1. p. 154. See Kersten. Reinf. Conc. Const-n. Part I.

First in Fig. 369 is represented a cellar under a court made of tamped concrete. The maximum live load amounts to 2000 kil

$/m^2$ or an 8-tonnes wagon with $500 \text{ kil}/m^2$ for a crowd of men. For the heavy wagon was to be regarded as the maximum load $3000 \text{ kil}/m^2$, and therefore the thick concrete slab was preferred to the lighter and thinner reinforced concrete slab. A skylight at the end of the cellar provided for lighting it. ²

Note 2.p.154. For the lighting of cellars under courts is recommended also the occasional insertion of glass concrete ceilings, see p. 45.

A two-story cellar arrangement is shown by Fig. 370. Precautions against the entrance of ground water were unnecessary here.

145 According to Figs. 369, 370, followed the construction of the side walls of the cellar in tamped concrete. But if a road runs in the immediate vicinity of the wall, then will the horizontal earth pressure be increased in important measure, wherefore a construction of the wall in tamped concrete would no longer suffice. Fig. 371 exhibits the construction of such a wall in reinforced concrete and strengthened by ribs. On these rests a transverse beam, which at the same time serves as a curb for the sidewalk. According to Fig. 372, vertical Monier arches transfer the horizontal forces to the separate piers.

Similar structures are shown by Figs. 373, 374. According to Fig. 373 (Kell & Loser, Dresden), the ceiling is formed as a corbel slab at the show window. The enclosing reinforced concrete wall receives an insulation next the cellar by a brick wall $1/2$ brick thick, and on the top beam of the reinforced concrete wall is set a brick wall. According to Fig. 374, the cellar has a projecting granary, whose ceiling is arranged at the same height as the floor of the ground story, and also serves as a terrace.

The external and light shaft walls of the Paper Manufactory of the Köslin Co are shaped as ample retaining walls to receive a considerable earth pressure by a loading railway slope (see Section C, 3). Another light shaft construction in reinforced concrete is shown in Fig. 380.

146 If the cellar floor be only temporarily below the level of the ground water, then suffices a watertight slab floor of reinforced concrete. Fig. 376. For a stronger water pressure, the upward force is transmitted to the walls by inverted tunnel or cross vaults. The floor is then always to be leveled in

and repair of the insulation is opposed the disadvantage, that it may be raised by especially strong expansion.

Note 1. p. 158. See B & E. 1908. p. 130.

Likewise the walls must be protected from the direct contact with the ground water. According to Fig. 385 in the insulation of the external walls, the asphalt felt is replaced by Siebold's asphalt-lead plates. ¹ The insulation extends to 15 cm above the highest level of the ground water; at this height the walls have a cast asphalt insulation against ascending ground water, with an external coating of tar to the height of the floor. ¹ The sheet lead insulation bisects vertically the external supporting walls at the middle. More convenient in application, but easily exposed to injuries, is an insulating layer on the exterior as in Fig. 386. If the insulation be placed on the inside (Fig. 386 a), then must one again consider the possibility of its being torn off by a strong pressure of water. The advantage of permanent control from the cellar is often of slight importance.

Note 1. p. 159. Recently ruperoid (p. 29) has been much used, that for protection from injuries receives a rich coating of the heated cementing material.

C. Walls against pressure of Wind, Earth or Water. ²

Note 2. p. 159. Division walls are treated on p. 53 to 58; on reservoir and silo walls see Section III B and C on buttress and wing walls. Kersten. Bridges in Reinforced Concrete. Part I.

For discussion are detached walls, (enclosure and blind walls, target walls), retaining and lining walls, bank and quay walls, and further also reinforced concrete sheetpiling, the use of which frequently comes in consideration for shore walls.

1. Enclosing and Blind Walls.

Enclosing walls in reinforced concrete are now built more and more, since they are cheaper and more durable than brick walls. The latter must be built disproportionately thick in view of their architecturally subordinate purpose, and therefore require deep excavations and extensive masonry in the ground. Their construction may be of two kinds as in Fig. 387.

a. Construction as a slab (corbel slab) anchored in the foundation, suitable for small heights; t and v are the bearing and distributing rods.

b. Construction with anchored piers with filling slabs betw-

between them, adapted to greater heights of wall. Instead of continuous foundation masonry suffice ground footings for the piers placed at certain distances, while the wall panels between these support themselves.

Since the wind pressure may come from either side, a corresponding double reinforcement is necessary. For the panel slabs of the mode b of construction suffices a reinforcement at the middle if piers are set close. (Fig. 389).

A suitable form for a concrete wall is shown by Fig. 388. Wooden struts -- set according to the progress of the concreting in the form -- preserve the exact thickness of the wall. In placing the concrete only 2 or 3 boards are set above each other, in order to tamp more conveniently.

A reinforced brick wall according to the Lenmann system is shown by Fig. 389. For reinforcement serve slender rods in the mortar joints. The use of I-beams for piers may be seen in Fig. 390. Compare this with the floor construction according to Fig. 2.

At the building of the new Provincial Sanatorium at Stralsund, was built a wall 4.5 to 4.9 m high enclosing the space for interned criminals (Fig. 391). This is perfectly smooth on the side next the court, while on the exterior it is divided into plinth, piers and ridge. To maintain the wall vertically with certainty, about 50 cm below the surface of the ground and directly on the tamped earth ^{were} arranged special foundation braces.

Figs. 392, 393, show the treatment of a blind wall on the line Görz-Triest. The wall has the purpose of concealing the wagons at a racecourse. Thickness of the masonry wall throughout is 12 cm, height of piers 2.4 and 6.0 m. ²

Note 2.p.131. See Nowak, "Reinforced Concrete on the new line in Austria constructed by the K.K. Railway Direction. Wm. Ernst & Son. 1907.

Recently the different parts of reinforced enclosing walls have frequently been made in factories, in order to save the somewhat costly work of the forms, as well as in a given case to provide for enlargement of the enclosed area or of the wall by taking apart and resetting the wall. According to Fig. 394 the piers are grooved at each side and made separately from the panel slabs. The latter consist of reinforced concrete slabs 3 cm thick and 25 cm wide, inserted in the piers after setting.

occurs during freezing and thawing of the back filling mass.

In Figs. 405 to 414 are represented various forms of retaining walls. Attention is particularly called to the following:--

Fig. 407. "The aprons" extending into the ground oppose a slipping of the wall (for example in saturation of the ground beneath).

Fig. 409. Retaining wall with two horizontal slabs for a particularly great consideration of the back filling. The base slab is stiffened by longitudinal beams; the second slab is a arched.

166 Fig. 412. The load on the upper broad horizontal slab is transferred by ribs to the base slab.

Fig. 413. Quay wall with sheet piling and platform on piles, from which rises a retaining wall stiffened by ribs -- as a continuation of the sheet piling.

Fig. 414. Retaining wall with slightly extended base but strong anchoring. The upper part of the wall may be inclined backward -- as shown. (Fig. 431).

In deciding on the reinforcement necessary in the different structural members, the following points of view are determinative:-- the wall has to receive the horizontal earth pressure, and thus is to be calculated for bending. (Distance between supports = distance between ribs). The horizontal bearing rods lie in the middle of the slab at the front side of the wall. Distributing rods extend vertically at distances of 10 to 30 cm and are connected with the bearing rods. The main ribs form slab beams, that are to be reinforced by tension rods. The chief purpose of the concrete in the ribs is twofold, the stiffening of the wall and base slab, and the protection of the tension rods. The small compression ribs at the front edge of the wall (Fig. 418, II) are to have compression rods at the inclined side. The foundation slab is supported by the ribs and is stressed in front by the earth pressure from beneath, at the back by the load or by the difference between the load and the earth pressure. Accordingly is to be made the arrangement of the steel reinforcement.

167 Some examples of the arrangement of the principal reinforcement are presented by Figs. 415 to 418. In Fig. 418 the bearing rods of the wall slab are omitted to obtain better supervision. Figs. 415 represents a construction of C. Brandt Co., Dusseldorf.

Note 1.p.169. Also see B & E. 1912. p.34; 1913. p.230; also Deuts. Bauz. 1908.Cem. Supp.p.91.

170 In hard ground the piles require shoes. Reinforcement is by longitudinal rods and bands; on the latter is to be placed special emphasis, as for piles. The completed driven sheet piling as a rule receives a cap at top by a horizontal beam. (Plate).

Further see the following Figs. 431, 433 to 434.

5. Walls against Pressure of Earth and Water.

(Ramparts, shore and quay walls). ¹

The advantage of reinforced concrete for the construction of shore walls lies first of all in its favorable behavior in varied depths of water. According to Fig. 430 concrete piles are driven at distances of 2.5 m, concrete slabs being set behind them. The piles are connected by a cap.

For a greater height of wall may be used tension anchors as in Fig. 431 (and 414), to prevent a separation toward the face.² In simpler cases suffice for the parts above water ^{-beams} I or reinforced concrete piers with slabs set between them (Figs. 431 a, b). The construction of an anchor slab in reinforced concrete is shown in Fig. 432.

Note 1.p.170. On the fastening of inclined walls see Section IV. (Figs. 519 to 522).

Note 2.p.170. Concerning the calculation of such walls, see among others, Zeits. f. Tiefbau. 1912. p. 138.

171 Notable is a Lock wall represented in Fig. 433 with full load of an inclined wall. ¹ Oblique piles and sheet piling are strongly combined with the covering slab into a rigid framework. The cross section of the sheet piles was previously shown in Fig. 426.

Note 1.p.171. Great ship canal, Berlin-Stettin. Lock at Hohensacken. Also see B & E. 1913. p.230.

For quay walls the sheet piling is placed behind instead of before, as in Fig. 434, being so far advantageous, since they can well receive the tensile stresses produced by the earth pressure in consequence of the great resistance of friction. ² In this way may be saved tension piles. Constructions with sheet piling in front are indeed usual; see Figs. 413, 435.

Note 2.p.171. Also see Deuts. Bauz. 1913. Cem. Supp. p.97.

framework, (Figs. 479 a, b, c, d) or is separated from this by a joint (Figs. 479 e, f). The arrangement last mentioned presents a greater danger for the tank remaining watertight, especially with unfavorable ground conditions.

The tank is almost always provided with an external protective covering against the unfavorable effects of varying weather. For simple cases suffices a layer of peat bark or sawdust, or even an adjacent wall of bricks or of corkboards. But most advantageous is an air circulation, that may be produced as in Fig. 480 by the addition of a separate enclosure in the form of a reinforced concrete or brick wall, of woodwork or a paving between reinforced concrete piers. Such a structure also presents an annular passage for occasional inspection of the exterior of the tank (Fig. 479); special ascending shafts as in Fig. 466 c are no longer necessary.

In Fig. 479 d is provided a strip float, which at the same time affords protection for the tank bottom from weather. Similar is the construction in Fig. 479 e; the tank here rests on a special platform, but is separated by a joint. Fig. 479 f shows the construction of a tank of 200 m^3 (Dittmar Wolfers & Co., Greifswald). Here also the tank is supported freely, indeed by means of an asphalt insulating layer 3 cm thick. The shaft of the tower consists of 8 piers. The arrangement of the roof is apparent in Fig. 485 c.

The construction of a tank of 120 m^3 capacity (Dyckerhoff & Widmann Co.) with internal shaft for ascent is to be seen in Figs. 481, 482, 483. Particularly Fig. 483 exhibits the reinforcement of the tank, and of the supporting ring-beam with the adjoining conical bottom of the enclosing wall. noteworthy here is the inclined position of the upper supports, for the purpose of dividing the circular curved supporting base into equal parts.

According to Fig. 484, the shaft of the tower consists of 4 separate piers, that support a platform for receiving a small tank (15 m^3). Here transverse cantilever beams serve for a uniform support of the bottom of the tank.

As covering is taken reinforced concrete in the form of rim conical or dome roofs (Fig. 485 a, b), but also wooden shingle and tile roofs, that by proper use of color may afford an effective artistic general impression.



The discharge can be properly so arranged, that it occurs directly before the retorts or boilers (Fig. 506). The side walls are mostly treated as retaining walls with buttresses. As an example of the reinforcement reference is made to Fig. 509. For the bottom are employed ribbed slabs, ³ and these are supported by columns, that are best received by continuous foundation slabs. (Fig. 512). To keep as much space free as possible, it is advisable to use spirally banded columns.

Note 8.p.201. Floors with ribs on top have the advantage, that the slabs at the points of maximum moments (one of the supports) properly lie beneath, and that the slabs are so far somewhat relieved by the ribs, since the solid masses rest chiefly on the ribs.

Further details are given by the coal elevators in Dankmarshausen represented in Figs. 510, 511, a construction of Robert Grastorf & Co, Hanover.

In smelting works the so-called ore pockets play an important part. According to Fig. 512 are provided four inclined bottom surfaces. The cross walls support at the middle a railway track and at the sides -- in vertical continuation -- are the truss piers of the roof. In another construction (Fig. 513) the floor beams extend into the interior; here also the elevated railway track extends directly above the cross wall. ¹

Note 1.p.202. Such an elevator, as shown in Figs. 512, 513, is built in great numbers by Weyss & Freytag Co; also see Handbuch für Eisenbeton. 2nd edition. Vol. 12. p. 78.

Allied to elevators are ore shoots, which serve to direct into the different bins the raw materials coming through a shaft to the railway, thus loading directly into the railway cars standing beneath. Figs. 514, 515 show a construction of R. Grastorf & Co, Hanover, indeed with fixed and movable ends.

Section IV. Applications in Hydraulic Engineering.

Dams serve to raise the water level and have the aim to obtain waterpower for industrial works, or irrigation of a domain lying above the natural water level. Likewise dams are employed for the canalization of rivers.

As for the construction in general, it is advisable to utilize the pressure of the water -- perpendicular to the resisting surface -- to increase the stability of the structure. Large bodies brought by the water then cannot injure the crown as

reinforcement is fixed to the ground by anchors in the earth. The anchors are placed by first boring a hole 4 cm diameter with a steel drill, then inserting a rod and finally filling the hole with cement mortar. The upper end of the rod is then connected with the steel network. Division joints at distances of 2.5 m prevent the formation of cracks.

A band protection (in Stettin) is composed of reinforced concrete blocks set together, and which in a given case may be reset again, is shown by Figs. 520, 521. The joints are later cast with cement. (See B & E. 1910. p.141).

A sea protection of reinforced concrete according to the Murralt system is represented in Fig. 522. Stepped beams have to support the covering slabs and are fastened by ground anchors a. The slabs themselves may be square; in smaller constructions as in Fig. 523, and after the setting is completed, may be fastened by anchors with conical heads.

107 Likewise for the regulation of rivers and in the construction of harbors and ship canals, reinforced concrete passes into increasing importance. Locks, strengthening bottoms of rivers, shore protections, moles, quay walls, landing piers, dykes, breakwaters, lighthouses, etc., built of reinforced concrete, present manifold practical and economical advantages. On quay walls see p. 170.

Fig. 524 exhibits the construction of a landing pier, where certain groups of piles are connected by struts. The piles are 2 and 3 m apart and are reinforced by four iron rods.

Fig. 525 shows the foundations of a sea pier with a double row of piers of hollow cylindrical section and enlarged footing. These wells, 1 which have a diameter of 2.5 m and walls 8 cm thick are filled with sand and covered by a layer of clay. Height of the well 8.75 m; distances between the wells lengthwise 5 m, crosswise 6.5 m. Cross beams 30 cm wide and at least 1.2m deep connect each pair of wells transversely and support a floor slab 32 cm thick. The facing at the land side is by a sheet piling wall. At the sea side of the pier is provided a protecting slab.

208 An example of the construction of quay walls by the use of floating blocks of reinforced concrete, moved into place by a steamboat, sank and then filled with concrete on sand, is presented in Fig. 526.

Recently reinforced concrete has also been utilized for the construction of pontoons and ships. The advantages it can offer are the following:— strength, elasticity and being watertight, no cost of maintenance and painting, durability, no growth of plants and barnacles, smooth exterior, built anywhere. Disadvantages are the great weight and the great draught required thereby. Larger ships have double sides.

Figs. 527, 528 show the longitudinal section and plan of such a pontoon for a river bath in Mannheim. There are 7 watertight divisions, each with a manhole in the deck. Calculation gave a cross section of the pontoon with 1.5 m width at bottom, 1.55 m at top, 1.27 m deep from the under side of bottom to the crown of the somewhat arched deck. The thickness of the bottom, side and deck were taken at 4.5 cm. Length of pontoon 10.3 m. Mixture of concrete 1 cement + 3 Rhine sand + 3 pumice gravel. 1

Note 1.p.209. For producing a watertight internal and external plastering was chosen an addition of ceresite, 10 p.c. of ceresite being added to the water for mixing.

Section V. Other Applications.

a. Agricultural and Forestry Buildings.

Buildings belonging here must first be fireproof, since they are distant from other dwellings, so that assistance at the right time during a fire is scarcely possible. The construction of such buildings in reinforced concrete proceeds rapidly, while bricks and split stone must often be brought several miles. Also the work of maintenance is only of the slightest kind.

Stables exhibit no rust -- and if porous materials are used for the concrete -- also no sweating of the walls, no condensation of water for a low external temperature. Reinforced concrete stables make possible the most careful cleanliness. For double rows of stalls are omitted the otherwise necessary columns in the middle passage, disturbing the passage and the work. If insulating ceilings are employed (p. 19), the stable odor does not penetrate into the attic, and thus does not injure the forage stored there. ²

Note 8-p. 302. If the construction of a hollow ceiling is impossible, then is advisable twice coating the tamped ceiling with tar, a layer of asphalt insulating felt, or a coat of melted asphalt 1 1/2 cm. thick, on this being finally placed a layer of lime cinder concrete 5 to 6 cm. thick. The dampness of the concrete cannot pass into the cinder concrete on account of the insulating layer, and as a bad conductor of heat, this again prevents any drip from the ceiling.

For barns, indeed both for new structures, as well as for covering old open field sheds, the reinforced concrete mode of construction is also well adapted. If also in most regions of Germany the cost of construction is somewhat more than for simple wooden sheds, they are always cheaper than masonry and half timber structures, yet presenting all the advantages of masonry buildings. Masonry and half timber barns are also objected to by many farmers on account of the particular disadvantages of defective ventilation, or the drying of the sheaves hauled in wet, which is entirely obviated by the thin walls, that permit sufficient circulation of air. Not to be forgotten is also the fact, that the barns are built fireproof and rapidly. Insurance premiums for the building as well as for the contents are also accordingly low.

Barn houses in the same kind of construction have double ex-

external walls with an insulating space 13 to 20 cm thick, that remains hollow or may be filled with bad conductors of heat like ashes, peat, sawdust, gravel or straw-loam.¹

Note 1.p.210. See p. 58.

Likewise boundary and fruit lattice walls are often built of reinforced concrete. They present the advantage, that the free walls between the piers allow the roots of the trees on the lattice to extend freely in all directions, thus making possible free passage for the roots. Enclosure walls were previously described on p. 159. Cheaper is the use of posts with wires stretched between them. The posts are made in factories, tamped in horizontal forms, and after sufficient hardening are set in the ground. The corner posts are to be braced in both directions. (Fig. 529).

Fig. 530 exhibits the construction of a vine post in reinforced concrete with a shaft 10 x 10 cm. The projecting arm has a hook for the stretched wire. For the case that the vine grows higher, there is opportunity to screw on a projecting rod.¹

Note 1.p.211. See B & E. 1912. p. 371.

Feeding troughs (Fig. 531) and watering troughs as well as stalls and partition walls of reinforced concrete are most durable, as well as manure pits, tanks, as well as tubs for water and for plants.

b. Street and Railway Construction.

Reinforced concrete ties have already been employed to a great extent. They are indeed heavy,² but do not require to be moved much, since they can be made at the place of use. They are extremely stiff, durable, resist weather, are also cheap in large quantities, and require neither plastering nor cost of repairs. Indeed it cannot be contested, that the making of a movable fastening for the rails is generally quite difficult, at the same time that ties of reinforced concrete show less elasticity in use.

Note 2.p.211. On the other hand the great weight indicates an increase of the stability of the track. Ties of reinforced concrete also thus afford great safety in use, in comparison with wooden or iron ties.

Reinforced concrete ties are to be recommended first for small railways. First the dimensions are less, the weight of tie is also less. Second, the stroke of the wheel is unimportant in consequence

of less velocity of travel, so that the disadvantage of less elasticity just mentioned rather disappears.

The tie of Dyckernoff & Widmann Co is represented in Figs. 532, 533, 534. It is 2.0 m long, 24 cm wide and 16 cm thick. At the middle of the tie the section is I-shaped. To fasten the rails serve creosoted wooden plugs in form of a frustum of a pyramid, that by their size allow no widening of the track. A spiral form of the plug has proved suitable. The kind of tie represented in the illustrations is calculated for a wheel load of 8 tonnes.

Another kind of tie frequently employed is the asbestos tie of H. Wölle Co; in this the cement concrete at the rail bearing is replaced by asbestos concrete.

Further see the Essay of Dr. Baetian, "New Experiences with Reinforced Concrete Ties," B & E. 1913. p. 78, 107.

A loading shed of factory made beams (Fig. 535) is shown by Fig. 535. The frames stand at distances of 3.0 m. As covering was employed tamped asphalt slabs.

The arrangement of cleaning pits (cleaning and examination pits) for car and locomotive sheds is evident from Fig. 536.

According to Fig. 537 as in Fig. 536, the rails rest on plates of reinforced concrete with connecting cross beams. Their distance apart lengthwise the rails is 1.90 m. The interval between each pair of rails is here covered by an easily removable planking. See B & E. 1909. p. 271.

In bad ground is recommended a box construction on piles with a filling of the interior with earth (Fig. 538). Cheaper in consequence of the simpler forms and smaller quantity of steel must be a construction with plain walls as in Fig. 5364.

Telegraph poles of wood suffer under the effect of the weather and thus have a short life. In consequence of the dampness of the earth the part in the ground rots, so that finally the entire remainder of the pole becomes useless, even if the wood be still sound. It is already better to coat the lower end of the wooden pole with tar, then to set it on a flat stone in a large hole and to enclose it by a layer of tamped concrete. Still better is the entire construction of the pole in durable and weatherproof reinforced concrete..

Fig. 539 shows the form of a telegraph pole on the Wölle system (Saxonia pole). The poles are generally made with rectangular cross section. Lengthwise and crosswise is arranged a rein-

reinforcement by round rods, while in the neutral zone are openings, that not only denote a saving of material, but also cause the pole to expose a smaller surface to the wind, and finally makes it possible to climb the pole without other aid. The masts are so set, that the greatest thickness lies in the direction of the greatest stress. They are made in wooden or iron forms, in a given case in the vicinity of the place of use.

Sieglwart (Zürich), as well as the Schleuder Works ¹, makes hollow poles in factories. The working machines permit any conical taper and any length. The thickness of the wall varies between 2.5 and 5.0 cm. Cross section is annular. The reinforcement consists of either mild steel or cast steel.

Note 1.p.214. On Schleuder concrete see p. 52.

A light pole in reinforced concrete is illustrated in Fig. 540. ² For later drawing in the electric cable a gas pipe is set in the longitudinal axis. The poles are dressed, the bungholes being rubbed level and polished on top. The arms for the arc lamps are made of wrought iron.

Note 2.p.214. For statical calculation see B & E. 1912.p.381.

Corbellings for widening streets beyond existing retaining walls are shown by Figs. 541, 542. The first illustration exhibits a construction in which the anchoring is in a mass of concrete lying behind the wall. This treats here of the widening of a street in Hanover, supported by a wall 4 m high. The anchoring is by 8 rods, each 10 mm diameter, (per metre in length). Mixing proportions of the corbel 1 : 2 : 3, of the block 1 : 3 : 4. The cost of the structure 35 m long, including railing, removal of part of wall, etc., amounted to 7300 marks.

Another widening of a street with an important projection of 5.6 m, is represented in Fig. 542. As a counterweight here serves a reinforced concrete anchor slab, that lies 5.0 m below the surface of the street. (Robert Wagner Street in Halle-a-S.). Deuts. Bauz. 1909. Gem. Supp. p. 97).

Curbs (stone borders) in reinforced concrete serve as substitutes for the more costly granite curbs. For protection against driving serve protecting steel shapes. Hollow blocks or those in ribbed form as in Figs. 543, 544, have the advantage of easy removal and setting.

c. Buildings for Sporting and Scientific Purposes.

Fig. 545 shows the treatment of a curved wall with included
akes for

stalls for wheels for the bicycle racecourse in Zürich. The slab is 6.0 cm thick. To connect the slope with the horizontal serves a cove at the bottom. In statical respects one has to do with a triangular frame with a cantilever arm. ¹ The inclination of the external slope is adapted to the most unfavorable resultant of the acting forces. In bad ground would naturally be required a tie-beam. The full sunshine makes necessary the arrangement of several expansion joints.

Note 1.p.216. On calculations see B & E. 1913. p.2.

Likewise for the creation of artificial hills in zoological gardens is well suited reinforced concrete as shown in Fig. 546. The illustration represents a frame for the tigers' den in the Budapest Zoological Garden; ² the corbel arm at the right is anchored in the retaining wall.

Note 2.p.216. See B & E. 1912. p.231.

According to similar basal ideas may also be constructed monuments in reinforced concrete. (Memorials).

d. Chimney Construction.

Chimneys of reinforced concrete present many advantages worthy of consideration over masonry chimneys. First is that the dead weight is substantially less, and also a smaller base is required. Wind and weather injure the exterior as little as the hot gases do the interior. Finally, also the durability of masonry chimneys is by far less, than that of reinforced concrete chimneys, that represent elastic beams resisting bending and fixed in the earth. ¹ No opening of joints occurs here. The durability of the chimney is also here ensured, even if the resultant of the dead weight and wind pressure acts outside a certain cross section. Fig. 547 shows the appearance of the recently erected chimney of a machine shop in Darmstadt. The otherwise usual dimension of the shaft is omitted; at 5.0 m height the shaft is held by the ceiling of the boiler house. ²

Note 1.p.217. For reinforced concrete chimneys by the resistance to bending a fivefold safety against breaking is easily obtained; for masonry chimneys one usually has but a twofold safety.

Note 2.p.217. See B & E. 1913. p. 281.

For greater heights chimneys mostly have double walls and consist of two concentric tubes of reinforced concrete. The outer tube has to receive the wind pressure and is strongly reinforced to correspond. The inner tube is so constructed, as

to be able to expand under the effect of the hot gases independently of the outer tube. ³

Note 3.p.217. By the usually one-sided introduction of the smoke gases is produced an unequal effect from heat in the vicinity of the inlet from the smoke flues. A uniform heating of the chimney walls occurs after the smoke gases have taken a central upward movement. Therefore extending the inner lining beyond a certain height is useless.

The vertical steel rods, that have to receive the tensile and bending stresses at the different sections of the chimney are uniformly distributed in the concrete in the form of slender rods, and are enclosed by horizontal rings at distances of about 50 cm.

e. Mining Construction.

The advantages of reinforced concrete for lining shafts and tunnels are rapid construction, watertight properties, good adherence to the rocks, great resistance with small thickness. Besides the shaft structures also come mining structures above ground, such as cooling towers, hoisting towers, buildings, and dust channels, etc., come in consideration.

Fig. 948 shows a shaft construction in reinforced concrete. The free space between the rock and the watertight wall of reinforced concrete 10 cm thick is filled with lean concrete. ¹

Note 1.p.218. Further see E & S. 1911. p.211.

$$x = \frac{25 \times 7.42}{100} \left(\sqrt{1 + \frac{200 \times 16.4}{25 \times 7.42}} - 1 \right) = 6.17 \text{ m.}$$

$$\sigma_e = \frac{125\,000}{7.42 \left(16.4 - \frac{6.17}{3} \right)} = 1175 \text{ kil/cm}^2.$$

$$\sigma_b = \frac{2 \times 125\,000}{100 \times 6.17 \left(16.4 - \frac{6.17}{3} \right)} = 28.2 \text{ kil/cm}^2$$

The moment of the other reinforcement is:—

$$M = 0.414 \frac{6.17^2}{8} \times 530 = 105\,000 \text{ cm kil.}$$

If in each joint be placed 1 rod 11 mm diameter and a rod 12 mm diameter, the steel section for 1 m width amounts to 6.89 cm².

$$h - a = 18 - 1.00 - 1.2 - \frac{1.2}{2} = 15.2 \text{ cm.}$$

Then $x = 5.7 \text{ cm}$, $\sigma_e = 1147 \text{ kil/cm}^2$, $\sigma_b = 27.7 \text{ kil/cm}^2$.

Example 2. Calculation for Window Lintel.

Floor 12 cm thick at the middle and 15 cm at the wall.

Load on lintel by floor = 1300 kil/m.

Clear span = 1.90 m and 2.10, averaging 2.0 m.

$$\text{Distance between supports } l = 2.0 + \frac{2 \times 0.25}{2} = 2.25 \text{ m.}$$

$$\text{Wall masonry} = 1.0 \times 0.90 \times 0.51 \times 1600 = 735 \text{ kil/m}$$

$$\text{Dead weight of lintel, estimated} = 405 \text{ ,,}$$

$$\text{Load of floor on lintel} = 1300 \text{ ,,}$$

Total,

$$2940 \text{ kil/m}$$

$$M = \frac{2940 \times 2.25^2}{8} \times 100 = \text{about } 18\,600 \text{ cm kil.}$$

The chosen section is evident in Fig. 550.

$$\text{Width for calculation} = b = \frac{51}{2} + \frac{225}{8} = \text{about } 60 \text{ cm.}$$

$$\text{Depth for calculation } h - a = \left(\frac{42 + 32}{2} = 37 \text{ cm} + 3 \right) = 34 \text{ cm.}$$

Selected are 4 rods of 12 mm each = 4.52 cm².

Example 3. Calculation of Footing of Pier. (See p. 51).

The axially acting compression $P = 50\,000 \text{ kil}$ (including dead load). The ground can only be loaded with 2.5 kil/cm², so that:

$$b = \frac{50\,000}{2.5} = \text{about } 150 \times 150 \text{ cm.}$$

$$\sigma = \frac{50\,000}{150^2} = 2.2 \text{ kil/cm}^2.$$

Fixing moment for $b = 1.0 \text{ m}$.

Load of one pier on ground is	55 000 kil.
Weight of footing = $1.0^2 \times 1.5 \times 2000 =$	3 000
$= 2.0^2 \times 0.5 \times 2000 =$	4 000
Total =	62 000 kil.

For a width of 2.0 m for footing:

$$\sigma_b = \frac{62\,000}{40\,000} = 1.55 \text{ kil/cm}^2.$$

The girts at a height of 6 m are arranged to better stiffen the supports, yet the piers are made safe for their entire length against buckling.

For practical reasons the girts are given the same dimensions as the columns (or piers).

242 Example 11. Calculation of an Angle Retaining Wall. (p.164).

The selected dimensions are shown in Fig. 573. The mass of earth and the back of the wall are both vertical. The ribs of the front and back footing slab are placed at distances of 8.5 m and are 20 cm thick.

Neglecting the ribs, the investigation is as follows.

$$\text{Weight I} = \frac{0.10 + 0.20}{2} \times 4.26 \times 2400 = 1\,534 \text{ kil.}$$

$$\text{Weight II} = 0.24 \times 2.0 \times 2400 = 1\,152$$

$$\text{Weight III} = 1.3 \times 4.26 \times 1600 = 8\,860$$

$$\text{Total vertical load} = 11\,546 \text{ kil.}$$

For calculating the horizontal resultant of the earth pressure the friction between wall surface and filling is neglected, and it is assumed, that the plane of cleavage coincides with the natural angle of repose (here 45°).

$$E = \frac{1}{2} \gamma h^2 \tan^2\left(\frac{90^\circ - \varphi}{2}\right) = \frac{1800 \times 4.26^2}{2} \times 0.173 = 2310 \text{ kil.}$$

$$\text{Then is } v \times 11\,546 = 1534 \times 0.62 + 1152 \times 1.0 + 8860 \times 1.35.$$

$$v = 1.22 \text{ m.}$$

$$w \times 11\,546 + 11\,546 \times 1.22 - 2310 \times 1.66 = 0.$$

$$w = 0.32 \text{ m.}$$

The point of application of the total resultant R accordingly lies within the kern, and the eccentricity is then:-

$$e = 100 - 82 = 18 \text{ cm.}$$

Maximum pressure on ground at front edge of footing:-

$$\sigma = \frac{11\,546}{200 \times 100} \left(1 + \frac{6 \times 18}{200} \right) = 0.83 \text{ kil/cm}^2.$$

This value corresponds to the allowable pressure on a soft clay ground or a very wet fine-grained sand. If a better ground exists, then one lessens the breadth of the footing. Weight





Section VII. Designing, Calculation of Quantities and Cost of a Workshop Floor with Girders and Piers.

In the following will be calculated and dimensioned a floor, consisting of floor slab, Girders, beams and supports. Determinative for the calculations are the Prussian Regulations of 1907. On account of the continuity of the floor slab and beams will be employed for calculating the bending moments, the Table given in the Appendix to Part I, 3rd edition. At the end of the calculation follows a graphical representation of the floor as designed with girders and piers, with the required steel reinforcements, as well as a calculation of the quantities, prices, and form of bid.

a. Calculation of Floor Slab.

The floor slab is regarded as a continuous beam on 5 supports: yet according to the official regulations, this connection can be extended only over 2 panels at most. Spans between the different supports = $l = 2.50$ m.

$$\text{Live load} = \quad \quad \quad = 500 \text{ kil/m}^2.$$

$$\text{Dead load (50 p.c.) } l = \quad \quad \quad = 250 \text{ ,,}$$

Note 1. p. 245. For structural members exposed to strong shocks or variations in loading, the live load increased by 50 per cent is to be employed in the calculations: Part I.

$$\text{Total} = \quad \quad \quad 750 \text{ kil/m}^2.$$

Assumed for the end panel a:-

$$\text{Dead load of floor 12 cm thick} = \quad \quad \quad 240 \text{ kil/m}^2.$$

$$\text{For plaster and flooring 4 cm thick} = \quad \quad \quad 100 \text{ ,,}$$

$$g_a = \quad \quad \quad 340 \text{ kil/m}^2.$$

Assumed for inside panels b:

$$\text{Dead load of floor 10 cm thick} = \quad \quad \quad 240 \text{ kil/m}^2.$$

$$\text{For plaster and flooring 4 cm thick} = \quad \quad \quad 100 \text{ ,,}$$

$$g_b = \quad \quad \quad 340 \text{ kil/m}^2.$$

The bending moments are:-

$$\text{End panel a: } M_{g_a} = 0.08 \times 340 \times 2.50^2 = + 194 \text{ m kil.}$$

$$M_p = 0.10 \times 750 \times 2.50^2 = + 469 \text{ m kil.}$$

$$M_{\max} = \quad \quad \quad + 663 \text{ m kil.}$$

$$\text{Inside panel b: } M_{g_b} = 0.025 \times 340 \times 2.50^2 = + 53 \text{ m kil.}$$

$$M_p = 0.075 \times 750 \times 2.50^2 = + 352 \text{ m kil.}$$

$$M_{\max} = \quad \quad \quad + 405 \text{ m kil.}$$

Middle pier (side girder):

$$-M_{g_a} = -0.100 \times 340 \times 2.50^2 = - 243 \text{ m kil.}$$

$$-M_p = -0.1167 \times 750 \times 2.50^2 = - 546 \text{ m kil.}$$

Determination of thickness of floor and the reinforcement.

Assumed is a crushing resistance of 240 kil/cm² for concrete. The allowable stresses for concrete are then for sixfold safety: $\sigma_b = 40$ kil/cm², and for steel $\sigma_e = 1000$ kil/cm².

and panel a: moment in panel = + 663 m kil.

For a stress $\sigma_b = 38$ kil/cm² and $\sigma_e = 1000$ kil/cm² are computed:—

$$h - a = 0.406 \sqrt{663} = 10.5 \text{ cm.}$$

$$h = 10.5 + 1.5 = 12.0 \text{ cm.}$$

$$f_e = 0.280 \sqrt{663} = 7.32 \text{ cm}^2 = 12 \text{ rods of 9 mm, with } f_e = 7.63 \text{ cm}^2.$$

Inside panel b: moment in panel = + 405 m kil.

For stress $\sigma_b = 36$ kil/cm² and $\sigma_e = 1000$ kil/cm²:—

$$h - a = 0.423 \sqrt{405} = 8.5 \text{ cm.}$$

$$h = 8.5 + 1.5 = 10.0 \text{ cm.}$$

$$f_e = 0.267 \sqrt{405} = 5.37 \text{ cm}^2, 9 \text{ rods of 9 mm, with } f_e = 5.72 \text{ cm}^2.$$

Middle support: (wide beams): moment at support = - 789 m kil.

By the arrangement of arches h is increased to 20 cm.

For stresses $\sigma_b = 21$ kil/cm² and $\sigma_e = 1000$ kil/cm²:—

$$h - a = 0.659 \sqrt{789} = 18.5 \text{ cm: } h = 18.5 + 1.5 = 20 \text{ cm.}$$

$$f_e = 0.166 \sqrt{789} = 4.66 \text{ cm}^2.$$

By the bent rods of the adjacent floor slab f_e is abundantly provided for.

b. Calculation of Side Beams.

The beams are to be regarded as continuous beams over 4 supports.

Span $l = 6.0$ m.: width B of load = 2.50 m.

Loading by live load and additions $= p = 750 \times 2.5 = 1875$ W/m².

Loading by floor panel a, $383 \times \frac{2.5}{2} = 485$..

Loading by floor panel b, $340 \times \frac{2.5}{2} = 425$..

Loading by weight of beam (estimated) $0.25 \times 0.45 \times 2400 = 270$..

$$q = 1130 \text{ ..}$$

The bending moments are:—

$$\text{End panel Ia, } M_g = 0.08 \times 1130 \times 6.00^2 = 3400 \text{ m kil.}$$

$$M_p = 0.10 \times 1875 \times 6.00^2 = 6750 \text{ m kil.}$$

$$M_{\max} = 10150 \text{ m kil.}$$

$$\text{End panel Ib, } M_g = 0.08 \times 1130 \times 6.00^2 = 3400 \text{ m kil.}$$

$$\text{Middle panel, Ia, } M_g = 0.025 \times 1130 \times 6.00^2 = 1060 \text{ m kil.}$$

$$M_p = 0.075 \times 1875 \times 6.00^2 = 5060 \text{ m kil.}$$

$$M_{\max} = 6120 \text{ m kil.}$$

$$\text{Middle support: } - M_g = -0.100 \times 1130 \times 6.00^2 = -4250 \text{ m kil.}$$

$$- M_p = -0.1167 \times 1875 \times 6.00^2 = -7870 \text{ m kil.}$$

$$M_{\max} = -7210 \text{ m kil.}$$

Determination of Bead Section and Reinforcement.

As limiting values of the allowable stresses are also here $\sigma_b = 40 \text{ kil/cm}^2$ and $\sigma_e = 1000 \text{ kil/cm}^2$. The loaded width b can be assumed at $1/5$ by the Prussian Regulations. Yet to work more advantageously, it is advisable to assume $b = h$ up to $2 h$, by which the section of the concrete is indeed increased, but the steel section is considerably reduced.

End panel Ia: end panel moment = + 10 150 m kil.

For stresses of $\sigma_b = 40 \text{ kil/cm}^2$ and $\sigma_e = 1000 \text{ kil/cm}^2$, and a compression width of $b = 2 h$:-

$$h - a = 0.39 \sqrt{\frac{10150}{0.90}} = 41.5 \text{ cm} : h = 41.5 + 3.5 = 45 \text{ cm.}$$

$$f_e = 0.293 \sqrt{10150} = 23.0 \text{ cm}^2, \text{ 7 rods of 23 mm, } f_e = 23 \text{ cm}^2. \quad 1$$

Note 1. p. 247. If b be assumed = $2 \times 50/6 = 16.6$, then:

$$h - a = 0.39 \sqrt{\frac{10150}{2.0}} = 27.8 \text{ cm} : h = 27.8 + 4.2 = 32.0 \text{ cm.}$$

$$248 \quad f_e = 0.293 \sqrt{10150 \times 2.0} = 41.8 \text{ cm}^2.$$

Middle panel Ib. Panel moment = + 6120 m kil.

The height h of the beam remains the same as for the end panel. Since the moment M is now considerably smaller, the stress in the concrete must be reduced.

For stresses of $\sigma_b = 29 \text{ kil/cm}^2$ and $\sigma_e = 1000 \text{ kil/cm}^2$, with a compression width of $b = 2 h = 0.90 m$:-

$$h - a = \sqrt{\frac{6120}{0.90}} = 41.5 \text{ cm} : h = 41.5 + 3.5 = 45.0 \text{ cm.}$$

$$f_e = 0.221 \sqrt{6120 \times 0.90} = 16.4 \text{ cm}^2 : \text{ 4 rods of 23 mm, } f_e = 16.6. \quad 1$$

Note 1. p. 248. Assuming $b = 2 \times 50/6 = 16.6$:-

$$h - a = \sqrt{\frac{6120}{2.00}} = 27.9 \text{ cm} : h = 27.9 + 4.1 = 32.0 \text{ cm.}$$

$$f_e = 0.221 \sqrt{6120 \times 2.0} = 24.5 \text{ cm}^2.$$

249 Middle pier (main girder). Moment at support = - 12120 m kil.

By the arrangement of arches h is increased to $45.0 + 20.0 = 65.0 \text{ cm}$. As compression width b is taken the width b_1 of the side beam = $0.25 m$.

For stresses $\sigma_b = 50 \text{ kil/cm}^2$ and $\sigma_e = 1000 \text{ kil/cm}^2$:-

$$x = \frac{15 \times 50}{1000 + 15 \times 50} \times 0.59 = 0.25 m.$$

$$M_g = 12.12 - 5 \times 0.25 \times 0.25 \times 50 (0.59 - 0.083) = 4.20 m \text{ tonnes.}$$

$$f_e = 4.8 \times \frac{1000 (0.59 - 0.25)}{1000 (0.25 - 0.083) (0.59 - 0.083)} = 14.15 \text{ cm}^2.$$

Bent upward are 4 rods of 23 mm diameter with $f'_e = 16.62 \text{ cm}^2$.

$$f_e = \frac{5000 \times 0.25 \times 0.25 \times 50}{1000} + \frac{1000 \times 4.2}{1000(0.59 - 0.06)} = 23.55 \text{ cm}^2.$$

Bent upward are 6 rods of 23 mm with $f_e = 24.93 \text{ cm}^2$.

The shear in the concrete is calculated at the support of the end panel Ia as follows:-

$$\text{Distance to zero line } x = 0.375 \times 41.5 = 15.6 \text{ cm.}$$

$$\frac{x}{3} = \frac{15.6}{3} = 5.2 \text{ cm.}$$

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$$h - a - \frac{x}{3} = 41.5 - 5.2 = 36.3 \text{ cm.}$$

$$\tau_o = \frac{V_{\max}}{b_1(h - a - \frac{x}{3})} = \frac{9165}{25 \times 36.3} = 10.1 \text{ kil/cm}^2.$$

The shear exceeding the limiting value of 4.5 kil/cm² must be received by bent rods.

$$c = \frac{\tau_o - 4.5}{\tau_o} \times \frac{1}{2} = \frac{10.1 - 4.5}{10.1} \times \frac{6.00}{2} = 1.66 \text{ m.}$$

Steel section to be bent is:-

$$f'_e = 3.5 c b_z (\tau_o - 4.5) = 3.5 \times 1.66 \times 0.25 \times 5.6 = 8.14 \text{ cm}^2.$$

2 rods of 23 mm are bent, with $f'_e = 8.30 \text{ cm}^2$.

Besides also a corresponding number of stirrups are arranged.

The bond stress between concrete and steel is computed for free support of the end panel Ia with 6 rods lying at bottom:-

$$\tau_1 = \frac{V_{\max}}{S(h - a - \frac{x}{3})} = \frac{9165}{5 \times 7.23 \times 36.8} = 7.0 \text{ kil/cm}^2. \quad 1$$

In consequence of the high bond stress a strong development of the end hooks is recommended.

Note 1. p.249. If for arches next beams stirrups are closely set, then the limiting value of $\sigma_b = 50 \text{ kil/cm}^2$ may be considered allowable. The new Swiss Regulations in this case even go to $\sigma_b = 70 \text{ kil/cm}^2$.

Note 1. p.250. This can be considered allowable. The "Preliminary Principles" of the German Concrete Union allow a limiting stress of 7.5 kil/cm². The Swiss Regulations generally neglect the bond stress.

The side beam I' is only loaded by the floor 10 cm thick. Yet since its $M_{\max}$ are not essentially smaller, it receives the same reinforcement as for the same concrete section.

c. Calculation of the Main Beam. (Beam II).

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